# Design optimization for fabrication of pretensioned concrete bridge girders: An example problem

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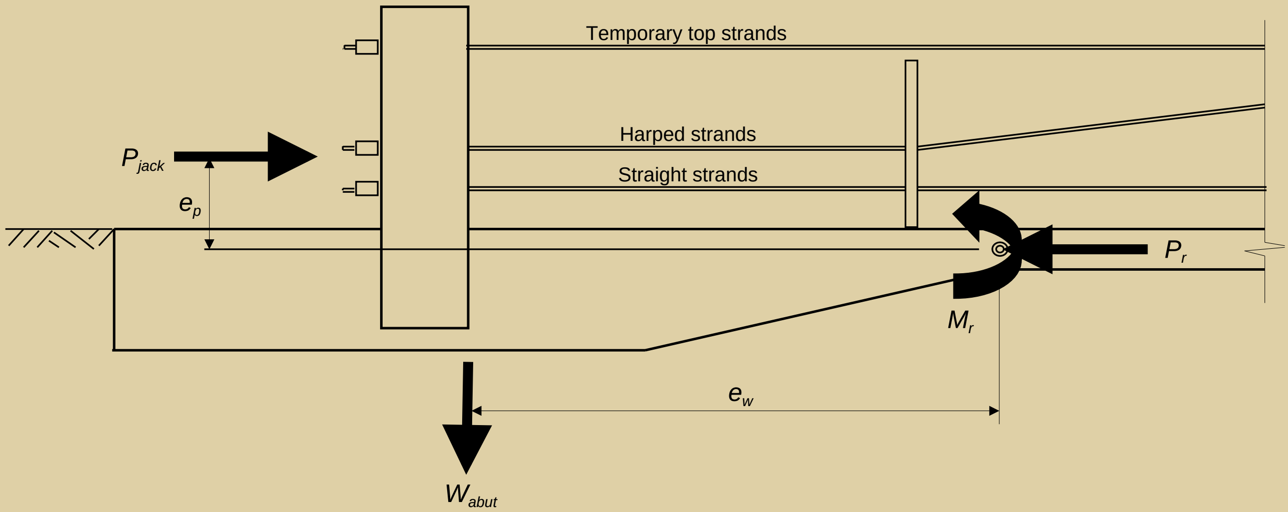
The use of high-strength concrete (HSC) and 0.6-in.-diameter (15.3 mm) strand in the fabrication of precast, pretensioned concrete bridge girders has resulted in improved economy through the use of longer spans, increased girder spacing or fewer girder lines, and shallower superstructures. However, the design presents the engineer with several challenges regarding fabrication, handling, shipping, and erection of long, slender girders. Seguirant presents techniques to overcome many of these challenges, such as the use of temporary top strands to improve lateral stability.[1]

While most of the difficulties in fabricating long-span HSC bridge girders have been overcome, some challenges remain. The primary issue is the capacity of existing stressing beds. Typically, these beds were not designed for the larger pretensioning forces and girder depths that are currently constructed. The increased forces and eccentricities at the girder ends combine to produce overturning moments and shear forces that quickly reach the capacity of existing beds. In addition, the greater concrete strengths required to strip the girders strain the fabricator's ability to maintain an efficient production schedule.

## Editor's quick points

- With the availability of higher-strength concrete, precasters must accommodate larger pretensioning forces and girder depths in their stressing beds.
- Washington State Department of Transportation (WSDOT) is updating its design methodology and detailing practices to facilitate optimized fabrication of precast, pretensioned concrete bridge girders.
- This paper presents a design procedure that optimizes the pretensioning configuration for maximum production efficiency.




$$\phi P_r \geq KP_{jack}$$

$$\phi M_r = \phi W_{abut} e_w \geq KP_{jack} e_p$$

**Figure 1.** This diagram shows the schematic of a typical stressing-bed abutment. Note: $e_p$ = eccentricity of prestressing force from center of bed overturning; $e_w$ = eccentricity of abutment weight from center of bed overturning; $K$ = selected factor of safety for stressing-bed design; $M_r$ = resisting moment of stressing-bed abutment; $P_{jack}$ = jacking force resisted by stressing bed; $P_r$ = axial resistance of stressing bed; $W_{abut}$ = weight of portion of stressing abutment resisting overturning; $\phi$ = strength-reduction or resistance factor.

Many precasting plants are responding to the challenge by installing greater-capacity stressing beds. However, fabricators frequently produce girders for many projects and customers simultaneously. To produce these girders in the most efficient manner possible, flexibility is needed to schedule different girder sizes and stressing requirements on available stressing beds. It is highly undesirable for a stressing bed to sit vacant because it does not have the capacity to produce a particular girder, when in fact it could be used if the design is optimized. Foregoing the use of the available stressing beds is clearly not productive and may adversely affect the cost and schedule of a particular project.

The Washington State Department of Transportation (WSDOT) is updating its design methodology and detailing practices to facilitate optimized fabrication of precast, pretensioned concrete bridge girders. The objectives are to reduce cost by saving on materials and labor, improve the schedule by optimizing plant usage and production efficiencies, and enhance quality by avoiding reinforcement congestion. Primary among these optimization techniques is the design and detailing of the permanent pretensioned-strand configuration. The goal is to determine the least required concrete strength at release and lifting while simultaneously placing the least possible demand on the stressing bed.

This paper presents a design procedure and example problem that optimizes the pretensioning configuration for maximum production efficiency while maintaining compliance with the applicable requirements for safe handling and shipping. The procedure provides manufacturers with a greater degree of flexibility in plant usage and production turnover. Other optimization techniques under examination by WSDOT are not specifically covered in this paper and can be found in Brice et al.[2]

Although the analytical and iterative procedures described in this paper may appear to be unnecessarily complex, a solution can be derived in a matter of minutes with properly designed software. With minimal effort, the additional design information provided in the contract documents can increase the field of bidders and lead to improved bid prices and schedules.

## Design considerations

### Stressing-bed capacity

Although many issues must be considered in the design of a stressing bed, its global capacity is largely dictated by two factors: the total force to be jacked and the height of that force above the floor. **Figure 1** shows a typical stressing abutment. Both the magnitude and height of the load contribute to overturning, which is resisted primarily by the dead weight of the abutment. While some components of a stressing bed can be reasonably strengthened to accommodate additional axial load, it is difficult to significantly increase overturning capacity. Typically, the lower the force is held above the floor, the greater the axial capacity of the bed.

The prestressing force and eccentricity required to satisfy final service conditions at midspan of a girder usually cannot be manipulated significantly. However, the height of the center of gravity of the pretensioning as it exits the girder can. Optimized design seeks to identify the lowest vertical exit location for the permanent pretensioning strands at the girder ends that does not adversely impact other aspects of the design.

Temporary top strands are frequently used to improve the lateral stability of long, slender girders.[1] They are also used to control stresses in shorter or shallower girders that are widely spaced in the structure. While these girders may not exhibit stability problems, they tend to be heavily prestressed with respect to their offsetting dead load.

It is most efficient to pretension these temporary top strands at the same time as the permanent strands. However, although their numbers are few, their eccentricity is large, and stressing beds often cannot handle their stressing in combination with that of the permanent strands. Consequently, optimized design allows these temporary strands to be installed in mono-strand ducts and post-tensioned after the permanent pretensioning force has been transferred into the girder. The stage of production at which these strands are stressed is left to the manufacturer's discretion, which provides added flexibility in the production process.

### Design for final service conditions

The most important aspect of design is obviously the performance of the girders in the completed structure. WSDOT designs pretensioned girders as simple spans for positive moment and allows no tension in the precompressed tensile zone under the AASHTO LRFD service III limit state[3]. This criterion almost always governs the amount of permanent pretensioning required in the girder. WSDOT also reinforces the cast-in-place concrete deck as if the spans were fully continuous. This design philosophy has historically resulted in durable bridges with significant overload capacity.

WSDOT employs harped strands in combination with straight strands to control concrete stresses at the ends of pretensioned girders and to contribute to the shear capacity of the section. While this practice raises the center of gravity of the pretensioned strands at the stressing abutments, it avoids the necessity of debonding strands. Debonded strands unnecessarily weaken the girder ends in flexure and shear and can cause cracking and other problems unless properly executed.

### Design for handling and shipping

The WSDOT *Bridge Design Manual*[3] and *Standard Specifications for Road, Bridge and Municipal Construction*[4] provide criteria for the analysis of handling and shipping (**Table 1**). These criteria are based on published literature, experience, and field measurements collected over time. Seguirant describes the calculation methods used in the analysis,[1] and the methods are demonstrated in the design example in this paper. All lateral stability calculations are based on the recommendations of Mast.[5,6]

While in the form, the girder spans end to end and its full dead-load moment is mobilized to counteract the effects of prestress at transfer. During lifting, the support locations are away from the girder ends by necessity, and the effectiveness of the dead-load moment in counteracting the prestress is reduced, or even reversed at the overhangs. Thus, the initial stresses in the girder are most severe when the girder is lifted from the form, and the required concrete strength at this stage is usually governed by the maximum concrete compressive stress.

Since 2006, the WSDOT standard specifications have required that prestress losses be calculated for evaluating stresses at every stage of construction using the refined method given in the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*.[7] Because the required concrete strength at release is not initially known and the losses depend on this strength, calculating concrete stresses and the corresponding required concrete strength is an iterative process.

For girders with harped strands, the concrete stresses during handling and shipping can be critical at one of three locations along the girder length:

- the end of the transfer length of the strand
- the lifting or support location
- the harp point

For long, slender girders, the lifting and support locations are dictated by requirements for adequate lateral stability. Larger overhangs at the girder ends result in improved sta-

**Table 1.** Washington State Department of Transportation criteria for handling and shipping

| | |
|---|---|
| Allowable temporary compressive stress | $0.6 f'_{ci}$ or $0.6 f'_c$ |
| Allowable tensile stress in plumb girder with no bonded reinforcement in the top flange | $0.095\sqrt{f'_{ci}} \le 0.200$ ksi |
| Allowable tensile stress in plumb girder with bonded reinforcement in the top flange to resist the total tension force in the concrete computed on the basis of an uncracked section | $0.190\sqrt{f'_{ci}}$ |
| Allowable tensile stress in tilted girder during shipping with no impact | $0.237\sqrt{f'_c}$ |
| Impact during handling | 0% |
| Impact during shipping (plumb girder only) | 20% |
| Maximum superelevation angle $\alpha$ | 6% |
| Sweep tolerance $e_{sweep}$, lifting | $^1/_{16}$ in./10 ft |
| Sweep tolerance $e_{sweep}$, shipping | $^1/_8$ in./10 ft |
| Lift device placement tolerance $e_{lift}$ | 0.25 in. |
| Position tolerance on truck $e_{truck}$ | 1 in. |
| Height of roll center above road $h_r$ | 24 in. |
| Distance from center of truck to center of dual tires $z_{max}$ | 36 in. |

Note: $f'_c$ = specified compressive strength of concrete; $f'_{ci}$ = required compressive strength of concrete at time of prestress transfer. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 ksi = 6.895 MPa.

bility but also increase the stresses in the concrete. Because the number and eccentricity of pretensioned strands at the harp point have been established in the design for final service conditions, the stresses at the harp point during lifting can be calculated. The stresses at both the transfer and lift points will depend on the exit eccentricity of the permanent pretensioning at the girder ends.

The optimized exit location for the permanent pretensioning results in the compressive stress at either the transfer or lift point being set about equal to the compressive stress at the harp point. This provides the lowest exit eccentricity that does not increase the required concrete strength at lifting. Finding this location is an iterative procedure. Experience has shown that a good starting point is a ratio of straight strands to harped strands of about 2:1. The exit eccentricity is varied by manipulating the harped strands.

The harped-strand exit location can be manipulated in one of two ways:

- lowering a fixed number of harped strands at the ends
- leaving the harped strands at their highest exit location and dropping pairs of strands from the harped-strand group into the straight-strand group—until the straight-strand group is full, if necessary

For a typical WSDOT girder, the top pair of harped strands exits 4 in. (100 mm) down from the top of the girder. Most standard WSDOT end details project several straight no. 7 (22M) mild-steel reinforcing bars from the web at the girder ends for shear-friction connections into the diaphragm (**Fig. 2**). Harped strands that are low in the web interfere with the placement of these bars. Consequently, fewer harped strands at a higher exit location provide the least potential for reinforcement congestion.

Temporary top strands improve lateral stability by reducing the tendency of a tilted girder to crack. For handling, they also reduce concrete stresses by helping to counteract the permanent pretensioning and by allowing the lifting embedments to be moved closer to the girder ends while still maintaining adequate stability. For shipping, temporary top strands allow for the large overhangs required for stability and turning-radius purposes. Most girders can be handled without temporary top strands, but many require them for shipping. It is generally beneficial to use the temporary top strands for handling if they will eventually be needed for shipping.

## Effect on camber

The WSDOT standard specifications currently define two levels of girder camber at the time the deck concrete is placed, denoted as $D_{40}$ and $D_{120}$. The concept is to provide the contractor with lower and upper bounds of camber that can be anticipated in the field.

The upper bound of camber $D_{120}$ is estimated at a girder

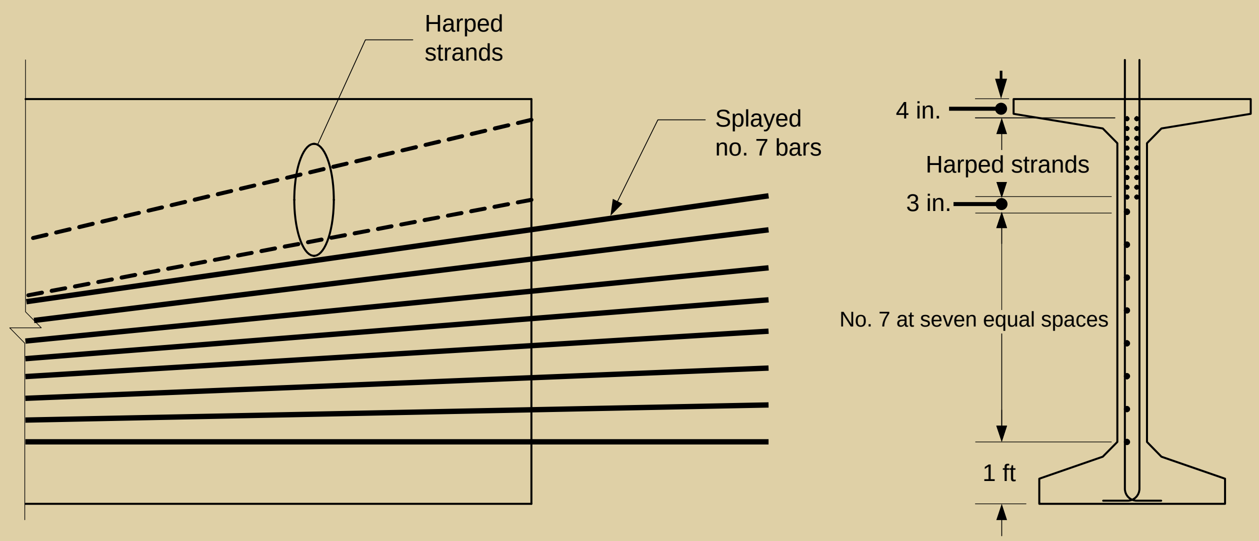



**Figure 2.** This diagram illustrates a typical Washington State Department of Transportation girder end detail with projecting no. 7 (22M) bars. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m.

age of 120 days after the release of prestress and is primarily intended to mitigate interference between the top of the cambered girder and the placement of concrete-deck reinforcement at midspan. It is also used to calculate the anticipated haunch height (distance from the top of the girder to the underside of the deck) at the girder ends and the height of the projecting stirrups. WSDOT projects stirrups straight from the top of the girder, and they are subsequently field bent 135 deg into the top layer of deck reinforcement. The age of 120 days was chosen because data has shown that additional camber growth after this age is negligible.

The lower bound of camber $D_{40}$ is estimated at a girder age of 40 days, or 30 days after the earliest allowable girder shipping age. To match the profile grade, girders with too little camber require an increased volume of haunch concrete along the girder length. For girders with large flange widths, such as the WSDOT WF-series girders, this can add up to significant quantities of additional concrete for a large deck placement. Thus, the lower bound of camber allows the contractor to assess the risk of increased concrete quantities and mitigates claims for additional material.

The confluence of camber and optimized design creates the issue that once a design is advertised and bid, it is a risky proposition to change the permanent pretensioning configuration, required concrete release strength, or temporary-top-strand requirements, all of which affect camber predictions. Changes to the $D_{40}$ and $D_{120}$ dimensions after the contract award will most likely affect costs and lead to requests for change orders. The time to optimize pretensioned girders for fabrication is during the design phase.

## Design example

This example problem applies to an actual WSDOT bridge project, spans 2 and 3 of the Interstate 5 (I-5) and Washington State Route 502 (SR 502) interchange, and demonstrates a step-by-step procedure for optimizing the design of pretensioned concrete girders for maximum production efficiency. The bridge consists of standard WF83G I-girders at 6 ft 9 in. (2.1 m) on center with an 8-in.-thick (200 mm) cast-in-place concrete deck. **Figure 3** shows a flowchart of the optimization process.

### Step 1: Design for final service conditions

With respect to pretensioning, the design for final service conditions establishes three essential pieces of information:

- the required concrete strength in service
- the required permanent pretensioning force (or number of strands) after all losses
- the eccentricity of the prestressing steel at midspan

This in-service design does not provide information on the required concrete strength at transfer of prestress or the eccentricity of the permanent pretensioning at the girder ends.

**Table 2** lists the data resulting from the girder design for final service conditions used in this example. Both ends of

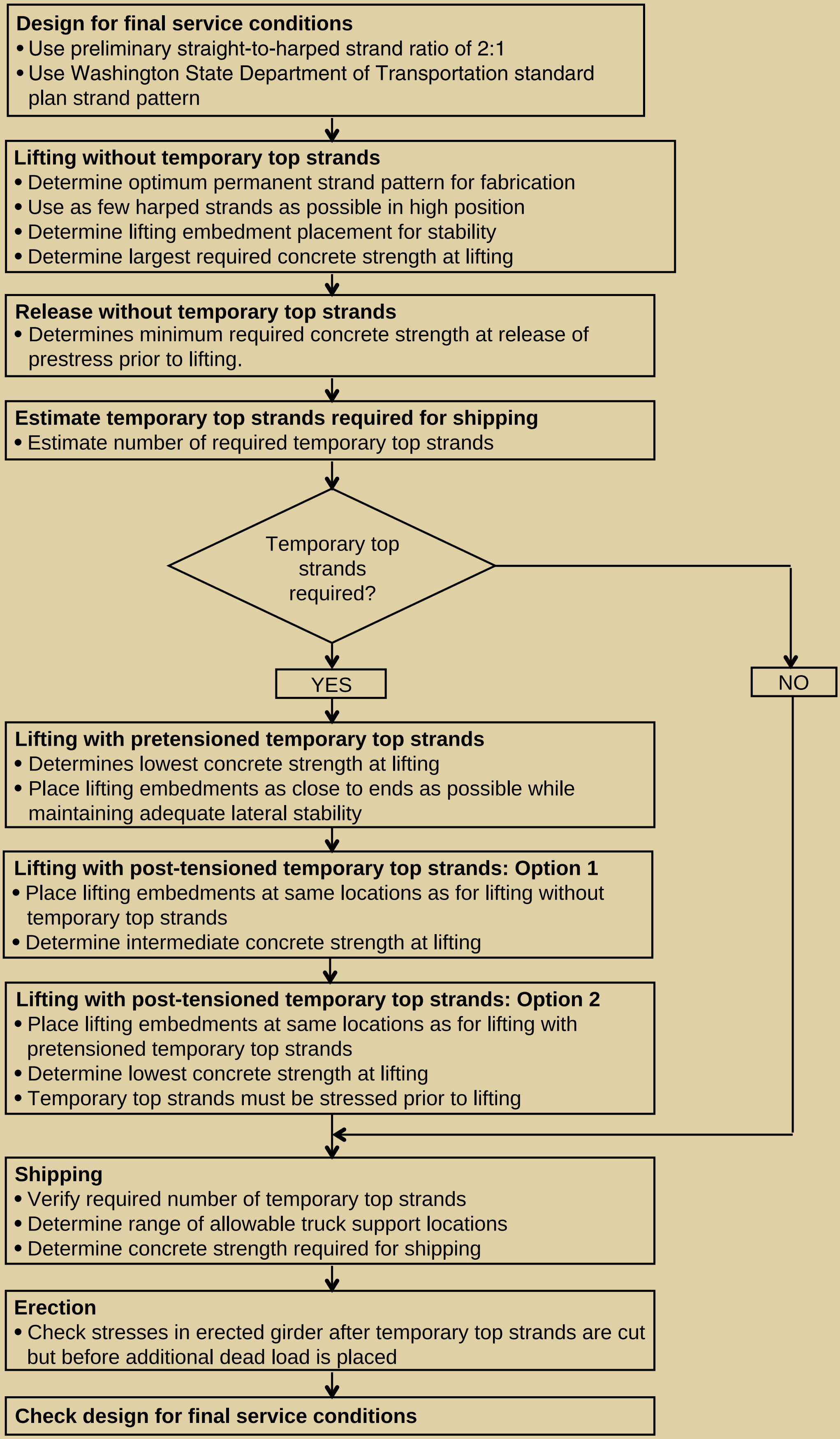



**Figure 3.** This flowchart can be used to optimize prestressed concrete girder design for fabrication.

**Table 2.** Results of design for final service conditions

| | |
|---|---|
| Overall length of girder $l_g$, ft | 175.50 |
| Clear span length center-to-center supports $l_{span}$, ft | 173.75 |
| Top-flange width $b_t$, in. | 49.02 |
| Bottom-flange width $b_b$, in. | 38.39 |
| Gross concrete area of girder cross section $A_g$, in.$^2$ | 972 |
| Major-axis moment of inertia $I_g$, in.$^4$ | 956,329 |
| Minor-axis moment of inertia $I_y$, in.$^4$ | 71,914 |
| Major-axis top-section modulus $S_t$, in.$^3$ | 22,230 |
| Major-axis bottom-section modulus $S_b$, in.$^3$ | 24,113 |
| Distance from CGC to girder top $y_t$, in. | 43.02 |
| Distance from CGC to girder bottom $y_b$, in. | 39.66 |
| Girder weight per unit length $w$, kip/ft | 1.114 |
| Girder weight $W$, kip | 195.5 |
| Volume-to-surface ratio $V/S$, in. | 3.16 |
| Design compressive strength of concrete $f_c'$, ksi | 8.5 |
| Density including reinforcement $\gamma_c$, kip/ft$^3$ | 0.165 |
| Density for elastic modulus calculations $w_c$, kip/ft$^3$ | 0.155 |
| Average relative humidity $H$, % | 80 |
| Area of one 0.6-in.-diameter strand $A_p$, in.$^2$ | 0.217 |
| Number of permanent 0.6-in.-diameter strands $N$ | 59 |
| Number of permanent straight strands | 38 |
| Number of permanent harped strands | 21 |
| Eccentricity of permanent strands at harp point $e_h$, in. | 35.90 |
| Initial jacking stress $f_{pj}$, ksi | 202.50 |
| Yield stress of strand $f_{py}$, ksi | 243.00 |
| Modulus of elasticity of strand $E_p$, ksi | 28,600 |

Note: CGC = center of gravity of bare concrete girder. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN; 1 ksi = 6.895 MPa.

these girders were configured in accordance with WSDOT standard end type D, in which the ends are cast 3 in. to 6 in. (75 mm to 150 mm) into a fixed crossbeam. The clear span $l_{span}$ is the distance between the oak blocks that temporarily support the girders at $10^1/_2$ in. (270 mm) from each end. This span is used to check the concrete stresses once the girders are erected and the temporary top strands are cut but before any additional dead load is placed on them. This can be critical for shorter span lengths and shallower girders at wide spacings, where the temporary top strands are used to control concrete stresses rather than provide lateral stability.

WSDOT specifies a standard compressive strength $f_c'$ for HSC of 8.5 ksi (59 MPa), unless a greater strength is required by design. Due to the simple span design for zero tension, the design concrete strength is normally not dictated by final service conditions. In many cases, the design concrete strength will be governed by concrete stresses during shipping, which will be examined later in this example.

The density of concrete $\gamma_c$ used to calculate the weight of WF-series girders has been found through calculations and measurements to be about 165 lb/ft$^3$ (25.9 kN/m$^3$), which is

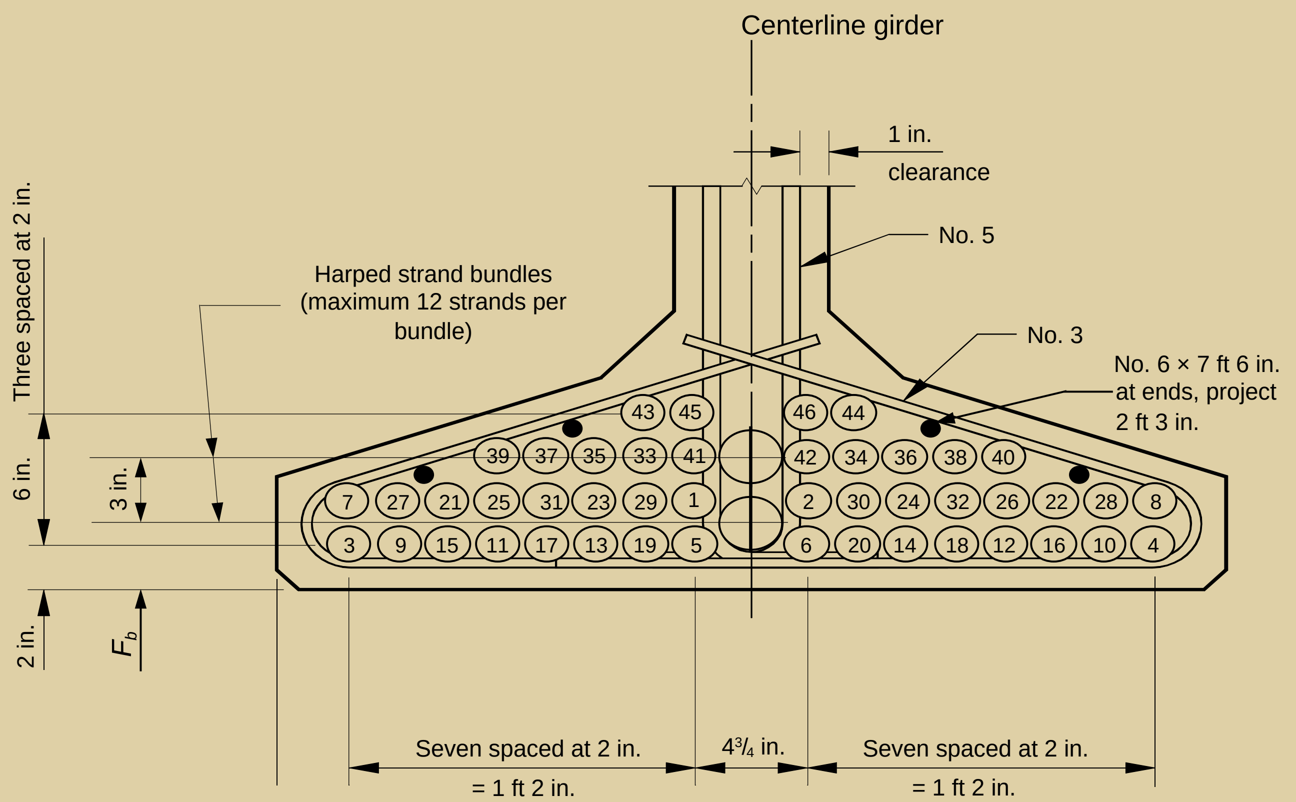



**Figure 4.** This drawing shows the standard Washington State Department of Transportation wide-flange-series girder strand pattern at midspan. The numbers on each strand represent the strand-pattern fill sequence. Note: $F_b$ = distance from bottom of girder to lowest harped-strand bundle at midspan. No. 3 = 10M; no. 5 = 16M; no. 6 = 19M. 1 in. = 25.4 mm; 1 ft = 0.305 m.

heavier than the 160 lb/ft$^3$ (24.4 kN/m$^3$) traditionally used by WSDOT for the density of concrete including reinforcement. This is due to the increased quantity of pretensioning and mild-steel reinforcement used in current designs compared with traditional designs.

The placement sequence of pretensioned strands at midspan is dictated by the WSDOT *Bridge Design Manual*[3] (**Fig. 4**). As a starting point, an algorithm places the strands at an approximate straight-to-harped ratio of 2:1. Strands are added until the final service conditions are satisfied, resulting in the required number and eccentricity of strands at midspan.

## Step 2: Check girder lifting without temporary top strands

Most girders can be lifted from the form without temporary top strands, but many require them for shipping. This step identifies the lifting locations that provide adequate lateral stability, and the corresponding concrete strength required for lifting without temporary top strands. For girders that do not require temporary top strands for shipping, this is the only handling check that is necessary. After step 4, the designer can skip steps 5 and 6 and proceed directly to step 7.

When temporary top strands are required for shipping but cannot be pretensioned with the permanent strands due to stressing-bed capacity, they must be post-tensioned. From the production perspective, it is most efficient to strip the girders without the temporary top strands and post-tension them in the yard later that day. This frees the stressing bed for continued production at the earliest possible time.

This step also establishes the optimized exit location of the permanent pretensioned strands at the girder ends.

**Find lifting-device location** Lifting-device locations that provide adequate lateral stability can be found iteratively or with the help of design aids. For a girder length of 175.5 ft (53.5 m) with 59 strands (**Fig. 5**), the lifting device locations are estimated to be about 12 ft (3.7 m) from each end, with a required concrete strength at lifting of 7.4 ksi (51 MPa). **Figure 6** shows the girder lifting configuration.

$a_l$ = length of overhang for lifting = 12 ft (3.7 m)

$l_l$ = girder length between lifting embedments = 151.5 ft (46.17 m)

$b$ = distance from end of girder to harp point

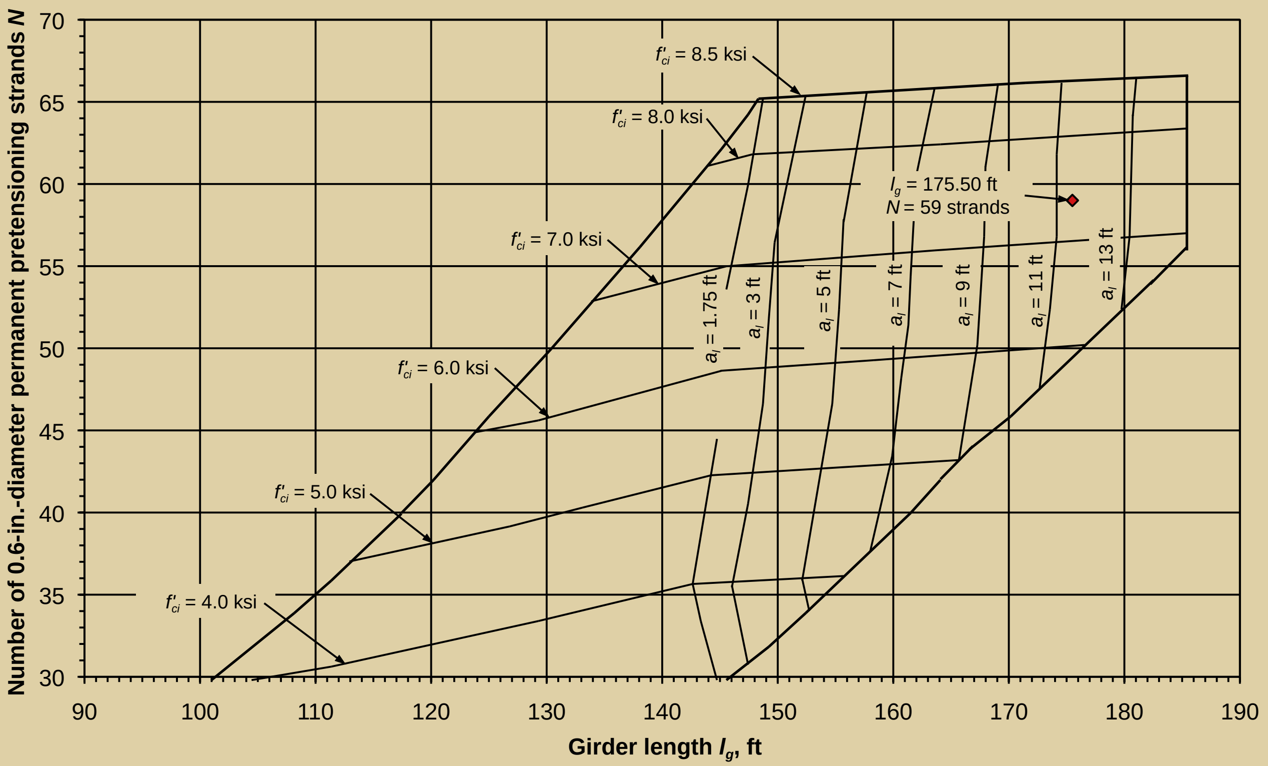



**Figure 5.** This graph shows WF83G girder handling without temporary top strands. Note: minimum $f'_{ci}$ of 4.0 ksi; minimum $a_l$ of 1.75 ft. $a_l$ = length of overhang for lifting; $f'_{ci}$ = required compressive strength of concrete at time of prestress transfer. 1 ft = 0.305 m; 1 ksi = 6.895 MPa.

$$= \frac{l_g}{2} - 0.1 l_g = \frac{175.5}{2} - 0.1(175.5) = 70.2 \text{ ft (21.4 m)}$$

$x_l$ = distance from lifting point or support to harp point = $b - a_l$ = 58.2 ft (17.7 m)

**Find optimum pretensioning configuration for lifting without temporary top strands** For lifting, the critical location for concrete stresses will be at the end of the transfer length, the lifting location, or the harp point. From the preliminary design for final service conditions, it is estimated that 38 straight and 21 harped strands will be required. WSDOT's standard strand-placement sequence results in an eccentricity of prestressing force at harp-point section $e_h$ of 35.90 in. (912 mm) for the middle 20% of the girder length. With this information, stresses at the harp point can now be calculated. As mentioned, this is an iterative process, only the last iteration of which is shown.

The jacking procedure in the plant accounts for losses in the stressing operation, so the stress in the pretensioned strand after seating is 202.5 ksi (1396 MPa). Relaxation losses between jacking and release, however, are not considered in the jacking operation. The strands are typically assumed to be released one day after jacking, and the resulting relaxation loss $\Delta f_{pR0}$ can be determined from the following equation.[8]

$$\Delta f_{pR0} = \frac{\log(24t_i)}{45}\left(\frac{f_{pj}}{f_{py}} - 0.55\right) f_{pj}$$

$$= \frac{\log[24(1)]}{45}\left(\frac{202.5}{243} - 0.55\right) 202.5$$

$$= 1.76 \text{ ksi (12.1 MPa)}$$

where

$t_i$ = assumed time between jacking and transfer of prestress

$f_{pj}$ = initial tensile stress in prestressing strands after jacking

$f_{py}$ = specified yield strength of prestressing steel

After release, the concrete shortens due to the induced prestressing force. It should be noted that WSDOT uses gross section properties for this analysis so that any elastic losses or gains must be calculated. When transformed section properties are used, elastic losses or gains are inherent in the stress calculations. The loss of prestress at a specific section due to elastic shortening $\Delta f_{pES}$ can be determined from AASHTO LRFD specifications Eq. (C5.9.5.2.3a-1).

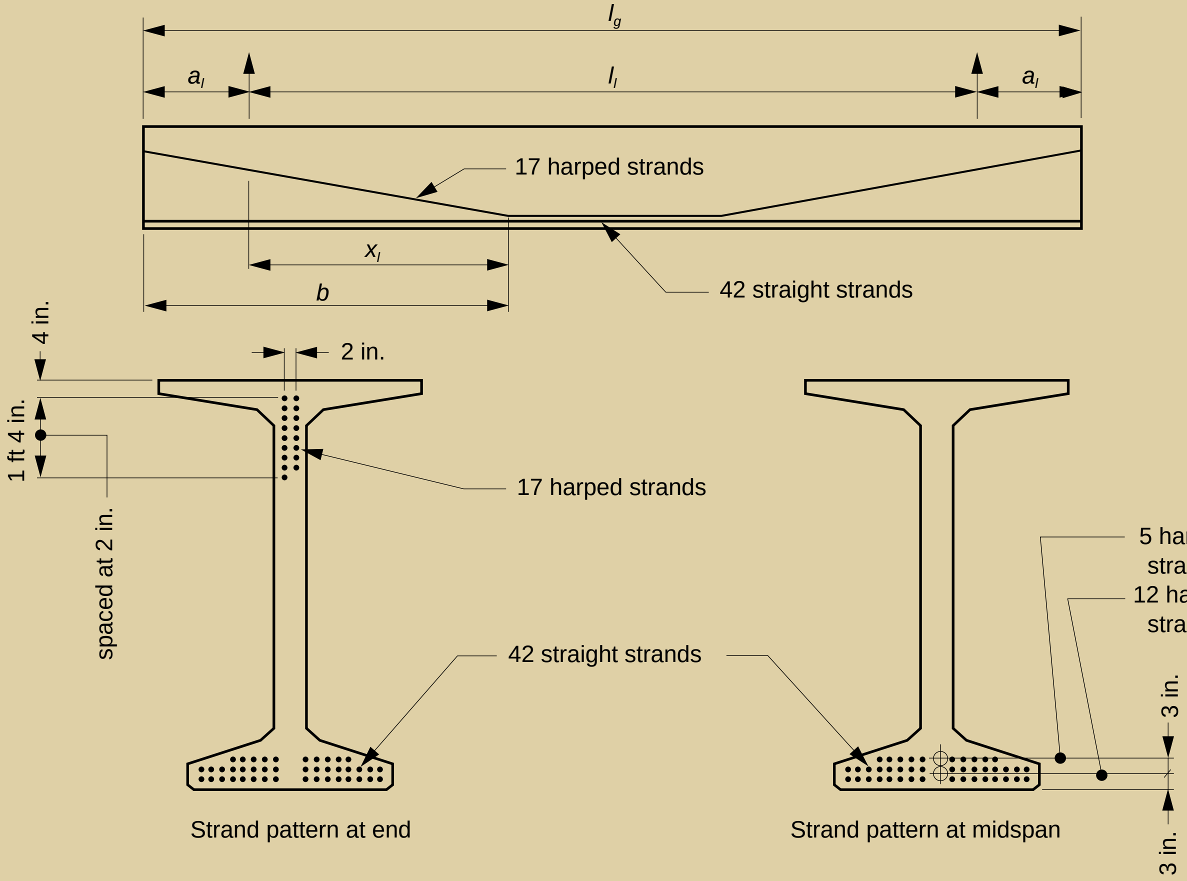



**Figure 6.** These diagrams illustrate the girder lifting configuration and optimized strand pattern. Note: $a_l$ = length of overhang for lifting; $b$ = distance from end of girder to harp point; $l_g$ = overall length of girder; $l_l$ = girder length between lifting embedments; $x_l$ = distance from lifting point or support to harp point. 1 in. = 25.4 mm; 1 ft = 0.305 m.

$$\Delta f_{pES} = \frac{A_{ps} f_{pbt}\left(I_g + e_p^2 A_g\right) - e_p M_g A_g}{A_{ps}\left(I_g + e_p^2 A_g\right) + \dfrac{A_g I_g E_{ci}}{E_p}}$$

where

$A_{ps}$ = total area of prestressing steel in concrete section = $A_pN$ = (0.217)(59) = 12.80 in.$^2$ (8258 mm$^2$)

$A_p$ = area of one prestressing strand

$N$ = total number of permanent pretensioning strands

$f_{pbt}$ = tensile stress in pretensioned strands immediately before transfer = $f_{pj} - \Delta f_{pR0}$ = 202.50 – 1.76 = 200.74 ksi (1384 MPa)

$I_g$ = gross major-axis moment of inertia

$e_p$ = eccentricity of prestressing force at section under consideration = $e_h$ = 35.90 in. (912 mm)

$A_g$ = gross area of concrete section

$M_g$ = self-weight bending moment of girder at section under consideration = $M_h$

$E_{ci}$ = modulus of elasticity of concrete at release strength

$E_p$ = modulus of elasticity of prestressing steel

At release, the girder cambers up in the form and is supported only at the ends. Therefore, $M_g$ is the dead-load moment at the harp point $M_h$ for a full-length simple span.

$$M_g = wb\left(l_g - b\right)\frac{12}{2} = 1.114\left(70.2\right)\left(175.5 - 70.2\right)\frac{12}{2}$$

= 49,397 kip-in. (5581 kN-m)

where

$w$ = weight per unit length of girder

$l_g$ = length of girder

The value of the required concrete strength at lifting is not known at this time. Assume that the required compressive strength of concrete at the time of prestress transfer $f'_{ci}$ is 7.4 ksi (51.0 MPa) as indicated in Fig. 5.

$$E_{ci} = 33{,}000 K_1 w_c^{1.5} \sqrt{f'_{ci}} = 33{,}000(1.0)(0.155)^{1.5}\sqrt{7.4}$$

$$= 5478 \text{ ksi } (37{,}771 \text{ MPa})$$

$K_1$ = factor to adjust for aggregate stiffness = 1.0

$w_c$ = density of concrete

The variable $K_1$ was found to exceed 1.0 in National Cooperative Highway Research Program report 496[9] for aggregates commonly used in western Washington. However, the value of 1.0 is used here because a larger study of aggregate stiffness has not been undertaken statewide. The value of 1.0 represents the national average.

$$\Delta f_{pES} = \frac{A_{ps} f_{pbt}\left(I_g + e_p^2 A_g\right) - e_p M_g A_g}{A_{ps}\left(I_g + e_p^2 A_g\right) + \dfrac{A_g I_g E_{ci}}{E_p}}$$

$$= \frac{12.80(200.74)\left[956{,}329 + (35.90)^2(972)\right] - 35.90(49{,}397)(972)}{12.80\left[956{,}329 + (35.90)^2(972)\right] + \dfrac{972(956{,}329)(5478)}{28{,}600}}$$

$$= 19.16 \text{ ksi } (132 \text{ MPa})$$

The value for $E_p$ of 28,600 ksi (197,200 MPa) represents a running average taken from mill certifications received by Concrete Technology Corp. over an extended period of time. Now that the initial prestress loss at the harp point has been determined, the stresses at the harp point during lifting can be calculated.

$$M_h = \frac{w}{2}\left(l_l x_l - x_l^2 - a_l^2\right)$$

$$= \frac{1.114}{2}\left[151.5(58.2) - (58.2)^2 - (12)^2\right](12)$$

$$= 35{,}324 \text{ kip-in. } (3991 \text{ kN-m})$$

where

$M_h$ = self-weight bending moment of girder at harp point

$P_{pt} = A_{ps} f_{pt}$ = (12.80)(202.5 – 1.76 – 19.16) = 2325 kip (10,342 kN)

where

$P_{pt}$ = prestressing force at section under consideration immediately after transfer

$f_{pt} = f_{pj} - \Delta f_{pR0} - \Delta f_{pES}$ = tensile stress in pretensioned strands at section under consideration immediately after transfer

$$f_t = \frac{P_{pt}}{A_g} - \frac{P_{pt} e_h}{S_t} + \frac{M_h}{S_t}$$

$$= \frac{2325}{972} - \frac{2325(35.90)}{22{,}230} + \frac{35{,}324}{22{,}230}$$

$$= 0.227 \text{ ksi } (1.57 \text{ MPa})$$

where

$f_t$ = concrete stress in top fiber of girder section

$S_t$ = major-axis top-section modulus

$$f_b = \frac{P_{pt}}{A_g} + \frac{P_{pt} e_h}{S_b} - \frac{M_h}{S_b}$$

$$= \frac{2325}{972} + \frac{2325(35.90)}{24{,}113} - \frac{35{,}324}{24{,}113}$$

$$= 4.388 \text{ ksi } (30.3 \text{ MPa})$$

where

$f_b$ = concrete stress in bottom fiber of girder section

$S_b$ = major-axis bottom-section modulus

Compression in the bottom flange governs at an allowable stress of $0.6 f'_{ci}$.

$$f'_{ci} = \frac{f_b}{0.6} = \frac{4.388}{0.6}$$

$= 7.31$ ksi (50.4 MPa) → use $f'_{ci}$ = 7.4 ksi (51.0 MPa)

The specified concrete strength at lifting is typically rounded up to the nearest 0.100 ksi (0.69 MPa) over the calculated strength. Because the assumed value of 7.4 ksi (51.0 MPa) for $f'_{ci}$ has been verified at the harp point, the optimization process can continue by establishing the lowest possible exit point for the pretensioning at the girder ends. This is the point at which the maximum compressive stress at either the end of the transfer length or the lifting point is approximately

**Table 3.** Lifting stresses without temporary top strands, 38 straight and 21 harped strands

| Point | $x$, ft | $e_p$, in. | $P_{pt}$, kip | $M_g$, kip-in. | $f_t$, ksi | $f_b$, ksi |
|---|---|---|---|---|---|---|
| Transfer | 3.00 | 13.80 | 2378 | -60 | 0.968 | 3.810 |
| Lifting | 12.00 | 16.76 | 2375 | -962 | 0.610 | 4.134 |
| Harp | 70.20 | 35.90 | 2325 | 35,324 | 0.227 | 4.388 |
| Midspan | 87.75 | 35.90 | 2329 | 37,382 | 0.317 | 4.314 |
| Harp | 105.30 | 35.90 | 2325 | 35,324 | 0.227 | 4.388 |
| Lifting | 163.50 | 16.76 | 2375 | -962 | 0.610 | 4.134 |
| Transfer | 172.50 | 13.80 | 2378 | -60 | 0.968 | 3.810 |

Note: $e_p$ = eccentricity of prestressing force at section under consideration; $f_b$ = concrete stress in bottom fiber of girder section; $f_t$ = concrete stress in top fiber of girder section; $M_g$ = self-weight bending moment of girder at section under consideration; $P_{pt}$ = prestressing force at section under consideration immediately after transfer; $x$ = distance from girder end to section under consideration. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN; 1 ksi = 6.895 MPa.

equivalent to the maximum compressive stress at the harp point. Beyond this point, if the exit eccentricity continued to lessen, the compressive stress at the ends would govern and increase the required concrete strength at lifting.

For the initial pattern of 38 straight and 21 harped strands, the top pair of harped strands is assumed to exit at 4 in. (100 mm) from the top of the girder. **Table 3** shows the stresses in the girder at lifting for this configuration.

Based on a comparison of the stresses at the three critical points, there is room to lower the center of gravity of the permanent pretensioned strands at the ends. Pairs of harped strands are lowered into the straight strand pattern until the stresses at the transfer point or lifting point approach those at the harp point. **Table 4** lists the resulting stress in this example for a ratio of 42 straight to 17 harped strands.

Although the maximum compressive stress at the lifting point slightly exceeds that at the harp point, the specified strength of 7.4 ksi (51.0 MPa) at lifting is unaffected. This is considered to be the optimum strand pattern for fabrication of this girder (Fig. 6).

## Check lateral stability during lifting without temporary top strands

$$e_{sweep} = \frac{1}{16}\left(\frac{175.5}{10}\right) = 1.10 \text{ in. (27.9 mm)}$$

where

$e_{sweep}$ = girder sweep tolerance during lifting, taken as $^1/_{16}$ in. per 10 ft (2 mm per 3 m) of girder length $l_g$

$$F_{offset} = \left(\frac{l_l}{l_g}\right)^2 - \frac{1}{3} = \left(\frac{151.5}{175.5}\right)^2 - \frac{1}{3} = 0.41$$

**Table 4.** Lifting stresses without temporary top strands, 42 straight and 17 harped strands

| Point | $x$, ft | $e_p$, in. | $P_{pt}$, kip | $M_g$, kip-in. | $f_t$, ksi | $f_b$, ksi |
|---|---|---|---|---|---|---|
| Transfer | 3.00 | 17.34 | 2362 | -60 | 0.585 | 4.132 |
| Lifting | 12.00 | 19.83 | 2361 | -962 | 0.280 | 4.410 |
| Harp | 70.20 | 35.90 | 2325 | 35,324 | 0.227 | 4.388 |
| Midspan | 87.75 | 35.90 | 2329 | 37,382 | 0.317 | 4.314 |
| Harp | 105.30 | 35.90 | 2325 | 35,324 | 0.227 | 4.388 |
| Lifting | 163.50 | 19.83 | 2361 | -962 | 0.280 | 4.410 |
| Transfer | 172.50 | 17.34 | 2362 | -60 | 0.585 | 4.132 |

Note: $e_p$ = eccentricity of prestressing force at section under consideration; $f_b$ = concrete stress in bottom fiber of girder section; $f_t$ = concrete stress in top fiber of girder section; $M_g$ = self-weight bending moment of girder at section under consideration; $P_{pt}$ = prestressing force at section under consideration immediately after transfer; $x$ = distance from girder end to section under consideration. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN; 1 ksi = 6.895 MPa.

where

$F_{offset}$ = offset factor that determines the distance between the roll axis and the center of gravity of the arc of a curved girder

$e_{lift}$ = lateral placement tolerance for lifting devices, taken as 0.25 in. (6.35 mm)

$e_i = e_{sweep}F_{offset} + e_{lift} = 1.10(0.41) + 0.25$
$= 0.70$ in. (17.8 mm)

$e_i$ = initial eccentricity of center of gravity of girder from the roll axis

Precast concrete downward deflection due to self-weight $\Delta_{self}$

$$\Delta_{self} = \frac{-5wl_g^4}{384E_{ci}I_g} = \frac{-5(1.114)(175.5)^4(12)^3}{384(5478)(956{,}329)}$$

$= -4.54$ in. (-115 mm)

Upward deflection due to prestress $\Delta_{ps}$

$e_e$ = eccentricity of total area of prestressing steel at end of girder = 16.52 in. (420 mm)

$e'$ = change in eccentricity of prestressing steel area between harp point and end of girder
$= e_h - e_e = 35.90 - 16.52 = 19.38$ in. (492 mm)

$P_{pt}$ = 2329 kip (10,360 kN) at midspan (Table 4)

$$\Delta_{ps} = \frac{P_{pt}e_e l_g^2}{8E_{ci}I_g} + \frac{P_{pt}e'}{E_{ci}I_g}\left(\frac{l_g^2}{8} - \frac{b^2}{6}\right)$$

$$= \frac{(2329)(16.52)(175.5)^2(12)^2}{8(5478)(956{,}329)} + \frac{(2329)(19.38)}{(5478)(956{,}329)}\left[\frac{(175.5)^2}{8} - \frac{(70.2)^2}{6}\right](12)^2$$

$= 7.83$ in. (199 mm)

Additional upward deflection due to girder overhang beyond lift points $\Delta_{ohang}$

$$\Delta_{ohang} = \frac{wa_l l_g^3}{16E_{ci}I_g} = \frac{(1.114)(12)(175.5)^3(12)^3}{16(5478)(956{,}329)}$$

$= 1.49$ in. (37.8 mm)

Total camber at lifting $\Delta$

$\Delta = \Delta_{self} + \Delta_{ps} + \Delta_{ohang} = -4.54 + 7.83 + 1.49$
$= 4.78$ in. (121 mm)

Adjusted $y_r = y_t - \Delta F_{offset} = 43.02 - (4.78)(0.41) = 41.05$ in. (1043 mm)

where

$y_r$ = height of roll axis above center of gravity of hanging girder

$y_t$ = height from top of girder to centroid of concrete section

$$\bar{z}_o' = \frac{w}{12E_{ci}I_yI_g}\left(\frac{1}{10}l_l^5 - a_l^2 l_l^3 + 3a_l^4 l_t + \frac{6}{5}a_l^5\right)$$

$$= \frac{1.114}{12(5478)(71{,}914)(175.5)}\left[\frac{1}{10}(151.5)^5 - (12)^2(151.5)^3 + 3(12)^4(151.5) + \frac{6}{5}(12)^5\right](12)^3$$

$= 17.37$ in. (441 mm)

where

$\bar{z}_0$ = theoretical lateral deflection of center of gravity of girder with full dead weight applied laterally

$I_y$ = gross minor axis (lateral) moment of inertia

$$\theta_i = \frac{e_i}{y_r} = \frac{0.70}{41.05} = 0.0171 \text{ rad}$$

where

$\theta_i$ = initial roll angle of a rigid girder measured from plumb

$$f_r = 0.237\sqrt{f_{ci}'} = 0.237\sqrt{7.4} = 0.645 \text{ ksi (4.45 MPa)}$$

where

$f_r$ = modulus of rupture of concrete

$f_t$ = 0.227 ksi (1.57 MPa) compression from Table 4

$b_t$ = top flange width = 49.02 in. (1245 mm)

$$M_{lat} = \frac{2\left(f_r + f_t\right)I_y}{b_t} = \frac{2\left(0.645+0.227\right)\left(71{,}914\right)}{49.02}$$

$= 2559$ kip-in. (289 kN-m)

where

$M_{lat}$ = lateral bending moment of girder at cracking

$$\theta_{max} = \frac{M_{lat}}{M_h} = \frac{2559}{35{,}324} = 0.0724 \text{ rad}$$

where

$\theta_{max}$ = tilt angle at which cracking begins measured from plumb

Compute factor of safety against cracking *FS*.

$$FS = \frac{1}{\dfrac{\bar{z}_0}{y_r} + \dfrac{\theta_i}{\theta_{max}}} = \frac{1}{\dfrac{17.37}{41.05} + \dfrac{0.0171}{0.0724}} = 1.52 > 1.0 \rightarrow \text{OK}$$

Compute factor of safety against failure *FS'*.

$$\theta'_{max} = \sqrt{\frac{e_i}{2.5\bar{z}_0}} = \sqrt{\frac{0.70}{2.5\left(17.37\right)}} = 0.1271 \text{ rad}$$

where

$\theta'_{max}$ = tilt angle at maximum factor of safety against failure measured from plumb

$$\bar{z}'_0 = \bar{z}_0\left(1+2.5\theta'_{max}\right) = \left(17.37\right)\left[1+2.5\left(0.1271\right)\right]$$

$= 22.90$ in. (582 mm)

where

$\bar{z}'_0$ = theoretical lateral deflection of center of gravity of girder with full dead weight applied laterally, computed using effective moment of inertia for tilt angle $\theta$ under consideration

$$FS' = \frac{y_r\theta'_{max}}{\bar{z}'_0\theta'_{max} + e_i} = \frac{\left(41.05\right)\left(0.1271\right)}{\left(22.90\right)\left(0.1271\right)+\left(0.70\right)} = 1.44 < 1.5$$

Mast[5,6] recommends a minimum factor of safety against cracking *FS* of 1.0 and a minimum factor of safety against failure *FS'* of 1.5. When *FS'* is less than *FS*, the maximum factor of safety occurs just before cracking. In this case, *FS'* should be taken to be equal to *FS*, so *FS'* is equal to 1.52. Without temporary top strands, the girder exhibits adequate stability with the lifting points at 12 ft (3.7 m) from the ends and a specified concrete strength $f'_{ci}$ of 7.4 ksi (51.0 MPa) at lifting. For girders requiring temporary top strands for shipping, this is the greatest strength that will be required for lifting.

## Step 3: Check girder stresses at release without temporary top strands

When the prestressing force is released, the girder cambers up in the form and spans end to end. This condition is less critical than when the girder is lifted because the dead-load moments at the critical locations more effectively counteract the prestressing force. The information provided by this step is the minimum concrete strength required for the release of prestress into a girder without temporary top strands before lifting.

In this case, the critical section is either at the end of the transfer length or at the harp point. The following example calculations are for the critical section at the end of the transfer length, which AASHTO LRFD specifications define as 60 strand diameters from the end of the girder or 36 in. (914 mm) for 0.6 in. (15.3 mm) diameter strand.

As in step 2, AASHTO LRFD specifications Eq. (C5.9.5.2.3a-1) can be used to calculate prestress losses due to elastic shortening. However, because the calculated concrete release strength will be lower for this condition than at lifting, a new value for $f'_{ci}$ of 6.7 ksi (46.2 MPa) is assumed.

$$e_t = e_e + \frac{e'}{b}x = 16.52 + \frac{19.38}{70.2}\left(3\right) = 17.34 \text{ in. (440 mm)}$$

where

$e_t$ = eccentricity of prestressing force at transfer-length section

$x$ = distance from girder end to section under consideration

$$M_g = wx\left(l_g - x\right)\frac{12}{2} = 1.114\left(3\right)\left(175.5-3\right)\frac{12}{2}$$

$= 3458$ kip-in. (391 kN-m)

$$E_{ci} = 33{,}000K_1w_c^{1.5}\sqrt{f'_{ci}} = 33{,}000\left(1.0\right)\left(0.155\right)^{1.5}\sqrt{6.7}$$

$= 5213$ ksi (35,944 MPa)

**Table 5.** Release stresses without temporary top strands

| Point | $x$, ft | $e_p$, in. | $P_{pt}$, kip | $M_g$, kip-in. | $f_t$, ksi | $f_b$, ksi |
|---|---|---|---|---|---|---|
| Transfer | 3.00 | 17.34 | 2352 | 3458 | 0.740 | 3.969 |
| Harp | 70.20 | 35.90 | 2314 | 49,397 | 0.866 | 3.777 |
| Midspan | 87.75 | 35.90 | 2319 | 51,456 | 0.956 | 3.703 |
| Harp | 105.30 | 35.90 | 2314 | 49,397 | 0.866 | 3.777 |
| Transfer | 172.50 | 17.34 | 2352 | 3458 | 0.740 | 3.969 |

Note: $e_p$ = eccentricity of prestressing force at section under consideration; $f_b$ = concrete stress in bottom fiber of girder section; $f_t$ = concrete stress in top fiber of girder section; $M_g$ = self-weight bending moment of girder at section under consideration; $P_{pt}$ = prestressing force at section under consideration immediately after transfer; $x$ = distance from girder end to section under consideration. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN; 1 ksi = 6.895 MPa.

$$\Delta f_{pES} = \frac{A_{ps} f_{pbt}\left(I_g + e_p^2 A_g\right) - e_p M_g A_g}{A_{ps}\left(I_g + e_p^2 A_g\right) + \frac{A_g I_g E_{ci}}{E_p}}$$

$$= \frac{12.80(200.74)\left[956{,}329 + (17.34)^2(972)\right] - 17.34(3458)(972)}{12.80\left[956{,}329 + (17.34)^2(972)\right] + \frac{972(956{,}329)(5213)}{28{,}600}}$$

= 17.00 ksi (117 MPa)

$$P_{pt} = A_{ps} f_{pt} = (12.80)(202.5 - 1.76 - 17.00) = 2352 \text{ kip } (10{,}462 \text{ kN})$$

$$f_t = \frac{P_{pt}}{A_g} - \frac{P_{pt} e_t}{S_t} + \frac{M_g}{S_t} = \frac{2352}{972} - \frac{2352(17.34)}{22{,}230} + \frac{3458}{22{,}230}$$

= 0.740 ksi (5.10 MPa)

$$f_b = \frac{P_{pt}}{A_g} + \frac{P_{pt} e_t}{S_b} - \frac{M_g}{S_b} = \frac{2352}{972} + \frac{2352(17.34)}{24{,}113} - \frac{3458}{24{,}113}$$

= 3.969 ksi (27.4 MPa)

**Table 5** gives the full results along the length of the beam. In this case, the stresses at the transfer point govern.

$$f'_{ci} = \frac{f_b}{0.6} = \frac{3.969}{0.6} = 6.62 \text{ ksi (45.6 MPa)} \rightarrow \text{use } f'_{ci}$$

= 6.7 ksi (46.2 MPa)

This minimum concrete strength can be used in one of two ways. First, for girders with post-tensioned temporary top strands, it ensures that allowable concrete stresses will not be exceeded in the interval between the release of the permanent pretensioning and post-tensioning of the temporary top strands prior to lifting. Second, this minimum strength can provide the manufacturer an option to release the permanent pretensioning at a concrete strength less than that required for lifting. In normal plant operations, once the cylinders have been tested for release, a one- to two-hour period is required to strip the forms and release the pretensioning prior to lifting. Significant additional concrete strength can be gained in this interval, though accelerated curing has been discontinued. This option can allow the crew to continue work and start on the next production cycle.

## Step 4: Estimate required number of temporary top strands for shipping

**Figure 7** indicates that for girders of this length and permanent prestressing level, four temporary top strands are marginally adequate if the shipped girders were made with concrete that had an $f'_c$ of 10.0 ksi (69.0 MPa). Because the specified design concrete strength is only 8.5 ksi (59 MPa), try six temporary top strands jacked to the same stress level as the permanent pretensioning.

Temporary top strands are typically distributed uniformly within the top flange, 2 in. (51 mm) below the top of the girder. Pretensioned temporary top strands are bonded for 10 ft (3 m) at each girder end and are unbonded for the remainder of the girder length in monostrand ducts. Post-tensioned temporary top strands use monostrand anchors at both ends and are unbonded in monostrand ducts for all but 3 ft (1 m) at the dead end (end away from the jacking end). Small foam blockouts are provided about 12 ft (3.7 m) from the jacking end so that the strands can be cut once the girder is erected and braced but before the concrete for the intermediate diaphragms is placed.

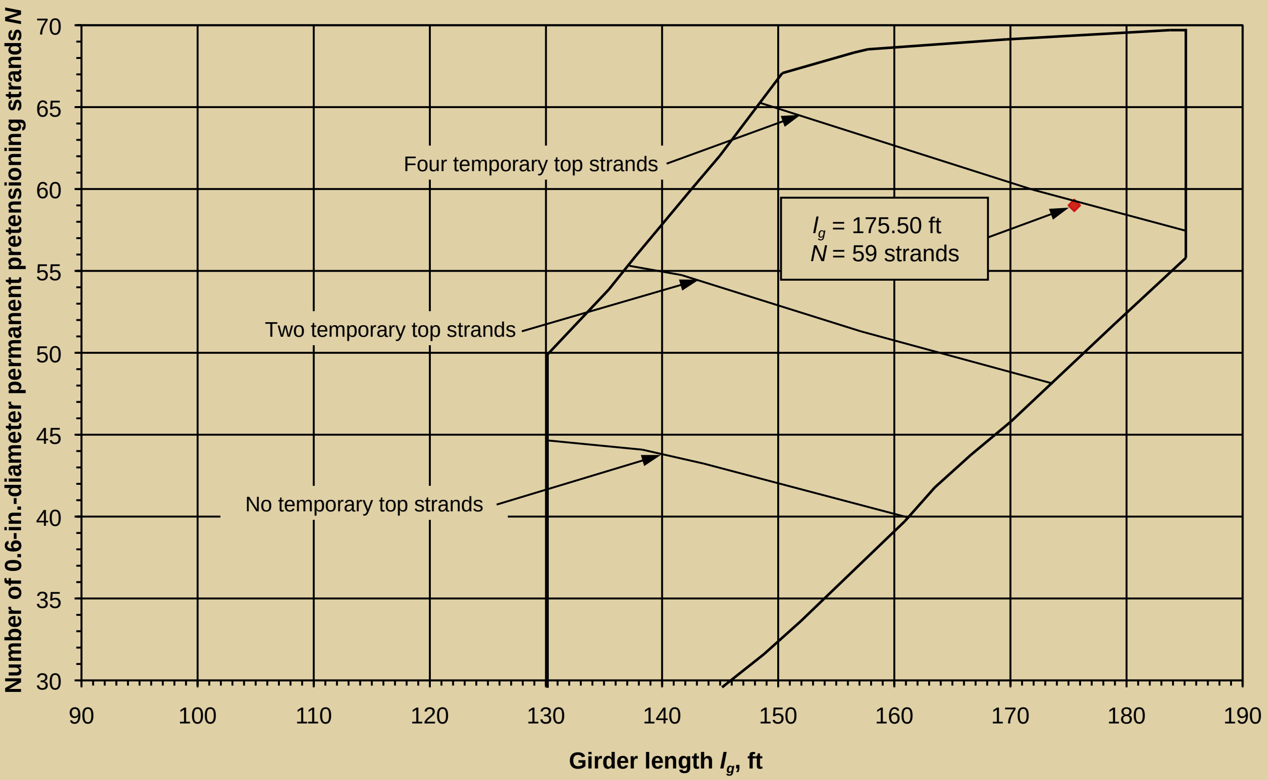



**Figure 7.** This graph shows the temporary-top-strand requirements for shipping. Note: 6% superelevation, 130 ft between supports, specified compressive strength of concrete $f'_c$ = 10.0 ksi (69 MPa). 1 in. = 25.4 mm; 1 ft = 0.305 m.

With design software capable of performing the lifting and shipping analyses simultaneously, the minimum number of temporary top strands can easily be determined by iteration. For cases where it is believed that no temporary top strands are necessary, proceed directly to the shipping analysis (step 7).

## Step 5: Check girder lifting with pretensioned temporary top strands

The calculations here are the same as in step 2 except that new values for the prestress amount, eccentricity, lifting locations, and concrete release strength are used. For six temporary top strands, the lifting locations appear to be about 9 ft (2.7 m) from the ends (**Fig. 8**) with a required strength at lifting of 7.0 ksi (48 MPa). For stresses at the harp-point section, try the following calculations.

$a_l$ = 9.5 ft (2.90 m)

$l_l$ = 156.5 ft (47.70 m)

$x_l = b - a_l = 70.2 - 9.5 = 60.7$ ft (18.50 m)

$A_{ps} = A_p(N + N_t) = (0.217)(59 + 6) = 14.11$ in.$^2$ (9100 mm$^2$)

where

$N_t$ = number of temporary top strands

$e_{pt} = 2 - y_t = 2 - 43.02 = -41.02$ in. (-1042 mm)

where

$e_{pt}$ = eccentricity of prestressing force in pretensioned or post-tensioned temporary top strands

$$e_{pi} = \frac{Ne_{pp} + N_t e_{pt}}{N + N_t} = \frac{(59)(35.90) + (6)(-41.02)}{(59 + 6)}$$

= 28.80 in. (732 mm)

where

$e_{pi}$ = eccentricity of combined prestressing force in permanent pretensioned strands and pretensioned or post-tensioned temporary top strands at section under consideration

$e_{pp}$ = eccentricity of prestressing force in permanent pretensioned strands

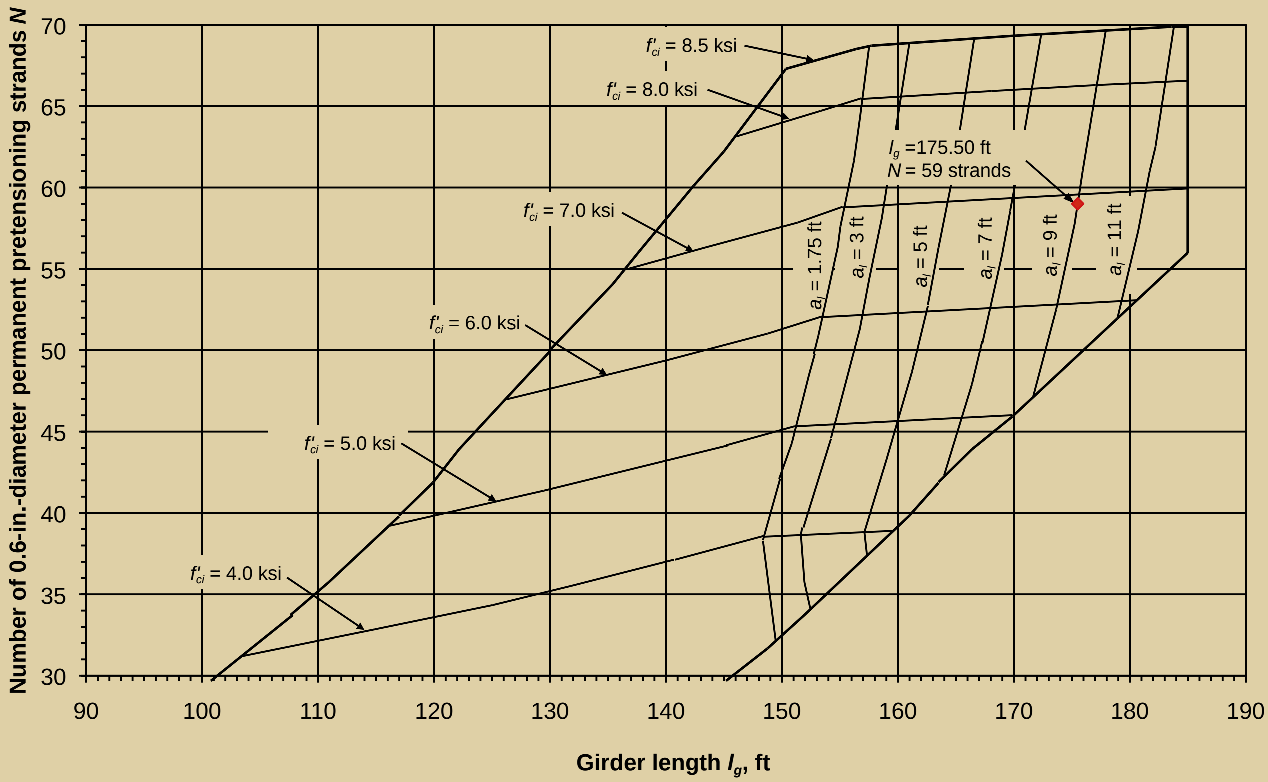



**Figure 8.** This graph shows WF83G girder handling with six temporary top strands. Note: minimum $f'_{ci}$ of 4.0 ksi; minimum $a_l$ of 1.75 ft. $a_l$ = length of overhang for lifting; $f'_{ci}$ = required compressive strength of concrete at time of prestress transfer. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 ksi = 6.895 MPa.

$$E_{ci} = 33{,}000 K_1 w_c^{1.5}\sqrt{f'_{ci}} = 33{,}000(1.0)(0.155)^{1.5}\sqrt{7.0}$$

$$= 5328 \text{ ksi } (36{,}740 \text{ MPa})$$

Substituting these values into the calculation procedure of step 2, **Table 6** lists the concrete stresses along the girder length. Compression in the bottom flange at the lift point governs:

$$f'_{ci} = \frac{f_b}{0.6} = \frac{4.168}{0.6} = 6.95 \text{ ksi (47.9 MPa)} \rightarrow \text{use } f'_{ci}$$

$$= 7.0 \text{ ksi (48.3 MPa)}$$

The required concrete strength at lifting of 7.0 ksi (48.3 MPa) is the least possible for this girder configuration. This is the most desirable of the lifting scenarios, but this scenario is not always possible due to capacity limitations of the stressing beds. This same scenario is also possible by placing the lifting devices at 9.5 ft (2.90 m) from the ends and post-tensioning the temporary top strands prior to lifting. However, this additional step delays the start of the next production cycle. Post-tensioned temporary top strands will be described in step 6.

The lateral stability factors of safety are also calculated by the same procedure in step 2 but by substituting the appropriate values of prestress amount, eccentricity, lifting locations, and concrete release strength for a girder with six temporary top strands. The resulting values of *FS* and *FS'* are 1.50 and 1.20, respectively. Because the factor of safety against cracking *FS* equals or exceeds the required margin for both cracking and failure, the lateral stability of the girder is acceptable.

## Step 6: Check girder lifting with post-tensioned temporary top strands

This step is necessary for cases where the stressing bed does not have the capacity to pretension the temporary top strands. Two options are available to the manufacturer for post-tensioned temporary top strands:

1. This is a discretionary step when the girder concrete has not achieved the required strength to lift the girder without temporary top strands. In this example, the lifting embedments are placed at 12 ft (3.7 m) from the ends (the same as for lifting without temporary top strands). After releasing the permanent prestress into the girder, post-tensioning of the temporary top strands will allow

**Table 6.** Lifting stresses with six pretensioned temporary top strands

| Point | $x$, ft | $e_{pi}$, in. | $P_{pt}$, kip | $M_g$, kip-in. | $f_t$, ksi | $f_b$, ksi |
|---|---|---|---|---|---|---|
| Transfer | 3.00 | 11.96 | 2603 | -60 | 1.275 | 3.970 |
| Lifting | 9.50 | 13.59 | 2602 | -603 | 1.060 | 4.168 |
| Harp | 70.20 | 28.80 | 2574 | 38,256 | 1.035 | 4.137 |
| Midspan | 87.75 | 28.80 | 2579 | 40,314 | 1.126 | 4.061 |
| Harp | 105.30 | 28.80 | 2574 | 38,256 | 1.035 | 4.137 |
| Lifting | 166.00 | 13.59 | 2602 | -603 | 1.060 | 4.168 |
| Transfer | 172.50 | 11.96 | 2603 | -60 | 1.275 | 3.970 |

Note: $e_{pi}$ = eccentricity of combined prestressing force in permanent pretensioned strands and pretensioned or post-tensioned temporary top strands at section under consideration; $f_b$ = concrete stress in bottom fiber of girder section; $f_t$ = concrete stress in top fiber of girder section; $M_g$ = self-weight bending moment of girder at section under consideration; $P_{pt}$ = prestressing force at section under consideration immediately after transfer; $x$ = distance from girder end to section under consideration. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN; 1 ksi = 6.895 MPa.

the girder to be lifted at a concrete strength between 7.0 ksi and 7.4 ksi (48 MPa and 51 MPa).

2. This is a means of lifting the girder at the least possible concrete strength of 7.0 ksi (48 MPa). In this case, the lifting embedments are placed at 9.5 ft (2.9 m) from the ends, and the temporary top strands must be post-tensioned prior to lifting to provide adequate lateral stability.

The post-tensioning of the temporary top strands will not have the same effect on prestress losses as pretensioned temporary top strands. The post-tensioned strands will not lose as much stress due to elastic shortening as the pretensioned strands but will experience prestress loss due to friction and live-end seating. In addition, the stressing of each individual strand will elastically influence the stress in the strands that have already been stressed, including the permanent strands.

**Figure 9** shows two potential cases for friction and seating loss along the length of the post-tensioned strands. In case 1, the girder is long enough that the seating loss is taken up within the girder length. In case 2, the seating loss affects the stress in the post-tensioned strands at the dead end of the girder. Because the tendons are straight, the friction loss is due to only wobble for either case.

$$\Delta f_{pF} = f_{pj}\left(1-e^{-\left(K_w l_g\right)}\right) = 202.5\left(1-e^{-\left[\left(0.0002\right)\left(175.5\right)\right]}\right)$$

$= 6.98$ ksi (48.1 MPa)

where

$\Delta f_{pF}$ = prestress loss due to friction along length of post-tensioned strand

$K_w$ = post-tensioning wobble friction factor, taken as 0.0002/ft (0.0007/m)

$$l_s = \sqrt{\frac{E_p\left(\Delta l\right)\left(l_g\right)}{12\Delta f_{pF}}} = \sqrt{\frac{28{,}600\left(0.375\right)\left(175.5\right)}{12\left(6.98\right)}}$$

$= 149.86$ ft (45.67 m)

where

$l_s$ = length over which anchorage seating affects prestressing force

$\Delta l$ = assumed seating slip at post-tensioning anchorages, taken as 3/8 in. (10 mm)

Because $l_s < l_g$, the case 1 prestress loss profile applies.

$$\Delta f_{pS} = \frac{2\left(\Delta f_{pF}\right)\left(l_s\right)}{l_g} = \frac{2\left(6.98\right)\left(149.86\right)}{175.5}$$

$= 11.93$ ksi (82.3 MPa)

where

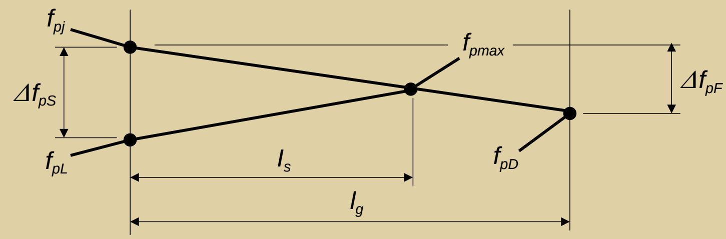
$\Delta f_{pS}$ = prestress loss at jacking end due to seating of post-tensioned strand

$f_{pD} = f_{pj} - \Delta f_{pF} = 202.5 - 6.98 = 195.52$ ksi (1348 MPa)

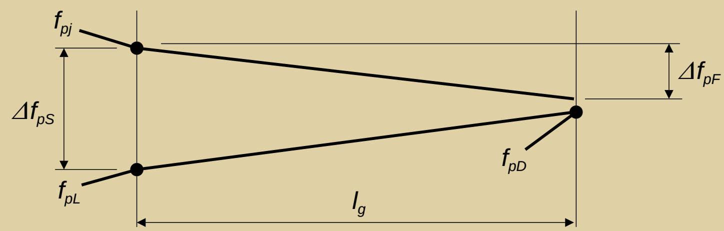
where

$f_{pD}$ = initial tensile stress in post-tensioned temporary top strand at dead end after seating

$f_{pL} = f_{pj} - \Delta f_{pS} = 202.5 - 11.93 = 190.57$ ksi (1314 MPa)




**Case 1: Post-tensioning profile**




**Case 2: Post-tensioning profile**

$$\Delta f_{pF} = f_{pj}\left[(l - e)^{-(Kl_g)}\right]$$

$$l_s = \sqrt{\frac{E_p(\Delta l)l_g}{12\Delta f_{pF}}}$$

$$\text{If } l_s \le l_g, \quad \Delta f_{pS} = \frac{2\Delta f_{pF} l_s}{l_g}$$

$$\text{If } l_s > l_g, \quad \Delta f_{pS} = \frac{E_p \Delta l + 12\Delta f_{pF} l_g}{12 l_g}$$

**Figure 9.** These diagrams show the post-tensioning friction-loss profiles. Note: $E_p$ = modulus of elasticity of prestressing steel; $f_{pD}$ = initial tensile stress in post-tensioned temporary top strand at dead end after seating; $f_{pj}$ = initial tensile stress in prestressing strands after jacking; $f_{pL}$ = initial tensile stress in post-tensioned temporary top strand at jacking end after seating; $f_{pmax}$ = maximum tensile stress in post-tensioned temporary top strand after seating but before elastic losses due to subsequent jacking; $K$ = selected factor of safety for stressing-bed design; $l_g$ = overall length of girder; $l_s$ = length over which anchorage seating affects prestressing force; $\Delta f_{pF}$ = prestress loss due to friction along length of post-tensioned strand; $\Delta f_{pS}$ = prestress loss at jacking end due to seating of post-tensioned strand; $\Delta l$ = assumed seating slip at post-tensioning anchorages.

where

$f_{pL}$ = initial tensile stress in post-tensioned temporary top strand at jacking end after seating

$$f_{pmax} = f_{pj} - l_s\left(\frac{\Delta f_{pF}}{l_g}\right) = 202.5 - 149.86\left(\frac{6.98}{175.5}\right)$$

= 196.54 ksi (1355 MPa)

where

$f_{pmax}$ = maximum tensile stress in post-tensioned temporary top strand after seating but before elastic losses due to subsequent jacking

Now that the initial stress profile of a post-tensioned temporary top strand is known, the effects on other existing strands can be determined. In the following calculations, the stresses at the harp-point section nearest the live end of the beam (where the strands are jacked) will be checked. This is for option 1, where the lifting embedments are 12 ft (3.66 m) from the ends. For this iteration, assume a required concrete strength at lifting of 7.1 ksi (49.0 MPa).

$$f_{ptmax} = f_{pL} + \frac{\Delta f_{pS}}{2}\left(\frac{b}{l_s}\right) = 190.57 + \frac{11.93}{2}\left(\frac{70.2}{149.86}\right)$$

= 193.37 ksi (1333 MPa)

where

$f_{ptmax}$ = tensile stress in first post-tensioned temporary top strand at section under consideration after seating

$P_{ptmax} = A_p f_{ptmax} = (0.217)(193.37)$
$= 41.96$ kip (187 kN)

where

$P_{ptmax}$ = prestressing force in first post-tensioned temporary top strand at section under consideration after seating

$e_{pt} = 2 - y_t = 2 - 43.02 = -41.02$ in. (-1042 mm)

$$E_{ci} = 33{,}000 K_1 w_c^{1.5} \sqrt{f'_{ci}}$$

$$= 33{,}000(1.0)(0.155)^{1.5}\sqrt{7.1}$$

$= 5366$ ksi (37,000 MPa)

The compressive stress in the concrete $f_c$ at the centroid of the temporary top strands is determined by the following equation.

$$f_c = \frac{P_{ptmax}}{A_g} + \frac{P_{ptmax} e_{pt}^2}{I_g}$$

$= 0.117$ ksi (0.81 MPa)

The loss of stress in the previously jacked strands is determined by the following equation.

$$\Delta f_{pt} = f_c \frac{E_p}{E_{ci}} = (0.117)\left(\frac{28{,}600}{5366}\right)$$

$= 0.624$ ksi (4.30 MPa)

where

$\Delta f_{pt}$ = elastic loss of stress in a temporary top strand due to post-tensioning of a subsequent temporary top strand

After the sixth strand is jacked, the stress in the first strand can be determined.

$f_{ptmin} = f_{ptmax} - 5\Delta f_{pt} = 193.37 - 5(0.624)$
$= 190.25$ ksi (1312 MPa)

where

$f_{ptmin}$ = tensile stress in first post-tensioned temporary top strand at section under consideration after jacking and seating of all temporary top strands

The average tensile stress $f_{ptave}$ in all six strands is then calculated.

$$P_{ptave} = \frac{193.37 + 190.25}{2} = 191.81 \text{ ksi (1323 MPa)}$$

$$P_{ptave} = N_t A_p f_{ptave} = 6(0.217)(191.81) = 250 \text{ kip (1112 kN)}$$

where

$P_{ptave}$ = average total prestressing force in post-tensioned temporary top strands at section under consideration after jacking and seating

The elastic effect of the temporary top strands on the permanent strands can now be calculated. This can be either a gain or an additional loss, depending on the eccentricity of the permanent strands. Because the resulting value is applied to the effective prestress, a gain is a positive value, while a loss is negative.

$$\Delta f_{pp} = \left(-\frac{P_{ptave}}{A_g} - \frac{P_{ptave} e_{pt} e_{pp}}{I_g}\right)\frac{E_p}{E_{ci}}$$

$$= \left[-\frac{250}{972} - \frac{250(-41.02)(35.90)}{956{,}329}\right]\left(\frac{28{,}600}{5366}\right)$$

$= 0.680$ ksi (4.69 MPa)

where

$\Delta f_{pp}$ = elastic change of stress in permanent strands due to post-tensioning of temporary top strands

The permanent prestress can be released into a girder sitting in the form with no temporary top strands at a concrete strength between 6.7 ksi and 7.1 ksi (46 MPa and 49 MPa). For handling purposes, the critical case is the strength at release that results in the least amount of elastic shortening loss because this case will result in the greatest remaining stress in the strand. Assume release at 7.1 ksi.

$$\Delta f_{pES} = \frac{A_{ps} f_{pbt}\left(I_g + e_p^2 A_g\right) - e_p M_g A_g}{A_{ps}\left(I_g + e_p^2 A_g\right) + \frac{A_g I_g E_{ci}}{E_p}}$$

$$= \frac{12.80(200.74)\left[956{,}329 + (35.90)^2(972)\right] - 35.90(49{,}397)(972)}{12.80\left[956{,}329 + (35.90)^2(972)\right] + \frac{972(956{,}329)(5366)}{28{,}600}}$$

$= 19.51$ ksi (135 MPa)

**Table 7.** Option 1 lifting stresses with six post-tensioned temporary top strands

| Point | $x$, ft | $e_{pi}$, in. | $P_{pt}$, kip | $M_g$, kip-in. | $f_t$, ksi | $f_b$, ksi |
|---|---|---|---|---|---|---|
| Transfer | 3.00 | 11.82 | 2600 | -60 | 1.290 | 3.951 |
| Lifting | 12.00 | 14.06 | 2601 | -962 | 0.988 | 4.232 |
| Harp | 70.20 | 28.45 | 2578 | 35,324 | 0.942 | 4.231 |
| Midspan | 87.75 | 28.44 | 2584 | 37,382 | 1.035 | 4.156 |
| Harp | 105.30 | 28.40 | 2581 | 35,324 | 0.947 | 4.230 |
| Lifting | 163.50 | 13.92 | 2607 | -962 | 1.006 | 4.227 |
| Transfer | 172.50 | 11.69 | 2606 | -60 | 1.309 | 3.946 |

Note: $e_{pi}$ = eccentricity of combined prestressing force in permanent pretensioned strands and pretensioned or post-tensioned temporary top strands at section under consideration; $f_b$ = concrete stress in bottom fiber of girder section; $f_t$ = concrete stress in top fiber of girder section; $M_g$ = self-weight bending moment of girder at section under consideration; $P_{pt}$ = prestressing force at section under consideration immediately after transfer; $x$ = distance from girder end to section under consideration. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN; 1 ksi = 6.895 MPa.

**Table 8.** Option 2 lifting stresses with six post-tensioned temporary top strands

| Point | $x$, ft | $e_{pi}$, in. | $P_{pt}$, kip | $M_g$, kip-in. | $f_t$, ksi | $f_b$, ksi |
|---|---|---|---|---|---|---|
| Transfer | 3.00 | 11.81 | 2598 | -60 | 1.290 | 3.948 |
| Lifting | 9.50 | 13.43 | 2599 | -603 | 1.077 | 4.147 |
| Harp | 70.20 | 28.44 | 2577 | 38,256 | 1.075 | 4.105 |
| Midspan | 87.75 | 28.43 | 2583 | 40,314 | 1.167 | 4.031 |
| Harp | 105.30 | 28.40 | 2579 | 38,256 | 1.080 | 4.104 |
| Lifting | 166.00 | 13.30 | 2605 | -603 | 1.095 | 4.142 |
| Transfer | 172.50 | 11.68 | 2605 | -60 | 1.308 | 3.944 |

Note: $e_{pi}$ = eccentricity of combined prestressing force in permanent pretensioned strands and pretensioned or post-tensioned temporary top strands at section under consideration; $f_b$ = concrete stress in bottom fiber of girder section; $f_t$ = concrete stress in top fiber of girder section; $M_g$ = self-weight bending moment of girder at section under consideration; $P_{pt}$ = prestressing force at section under consideration immediately after transfer; $x$ = distance from girder end to section under consideration. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN; 1 ksi = 6.895 MPa.

The tensile stress in pretensioned permanent strands at the section under consideration after transfer plus jacking and seating of all temporary top strands $f_{pp}$ can now be calculated.

$$f_{pp} = f_{pj} - \Delta f_{pR0} - \Delta f_{pES} + \Delta f_{pp} = 202.5 - 1.76 - 19.51 + 0.68 = 181.91 \text{ ksi (1254 MPa)}$$

The prestressing force in pretensioned permanent strands at the section under consideration after transfer plus jacking and seating of all temporary top strands $P_{pp}$ can be determined.

$$P_{pp} = A_{ps} f_{pp} = (12.80)(181.91) = 2328 \text{ kip (10,360 kN)}$$

$$P_{pt} = P_{pp} + P_{ptave} = 2328 + 250 = 2578 \text{ kip (11,470 kN)}$$

The eccentricity of combined prestressing force in permanent pretensioned strands and post-tensioned temporary top strands $e_{pi}$ can now be calculated.

$$e_{pi} = \frac{\left(P_{pp}e_{pp} + P_{ptave}e_{pt}\right)}{P_{pt}} = \frac{\left[2328\left(35.90\right) + 250\left(-41.02\right)\right]}{2578} = 28.45 \text{ in. (723 mm)}$$

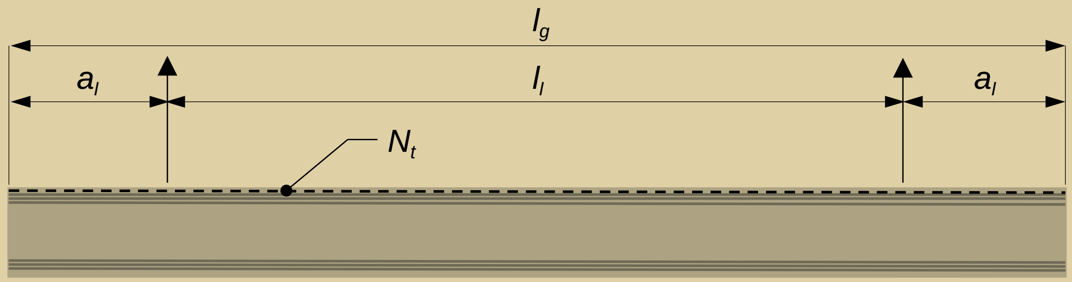



**Figure 10.** This diagram shows the concrete release strength and lifting summary. Note: $a_l$ = length of overhang for lifting; $l_g$ = overall length of girder; $l_l$ = girder length between lifting embedments; $N_t$ = number of temporary top strands.

$$f_t = \frac{P_{pt}}{A_g} - \frac{P_{pt}e_{pi}}{S_t} + \frac{M_h}{S_t}$$

$$= \frac{2578}{972} - \frac{2578(28.45)}{22{,}230} + \frac{35{,}324}{22{,}230}$$

= 0.942 ksi (6.50 MPa)

$$f_b = \frac{P_{pt}}{A_g} + \frac{P_{pt}e_{pi}}{S_b} - \frac{M_h}{S_b}$$

$$= \frac{2578}{972} + \frac{2578(28.45)}{24{,}113} - \frac{35{,}324}{24{,}113}$$

= 4.231 ksi (29.2 MPa)

Compression in the bottom flange governs:

$$f'_{ci} = \frac{f_b}{0.6} = \frac{4.231}{0.6} = 7.05 \text{ ksi (48.6 MPa)}$$

Therefore, use $f'_{ci}$ = 7.1 ksi (49 MPa)

**Table 7** gives the complete list of stresses for option 1 with six post-tensioned temporary top strands. **Table 8** lists the concrete stresses with six post-tensioned temporary top strands for option 2 with the lifting embedments at 9.5 ft (2.9 m) from the ends and a required concrete strength at lifting of 7.0 ksi (48 MPa).

When compared with Table 6 for lifting with pretensioned temporary top strands, the post-tensioned temporary top strands are slightly more effective in reducing the required strength at lifting. In this case, the pretensioned strands lose more stress due to elastic shortening than all of the losses in the post-tensioned strands combined. This may not be true of shorter girders in which the seating loss can affect the stresses in the tendons at the dead end of the girder. It is recommended that the governing case between the pretensioned and post-tensioned analyses be used to specify the required concrete strength at lifting.

From the lateral stability perspective, option 1 does not need to be checked. The lifting embedments have been placed for adequate stability without temporary top strands. The primary purpose of adding the temporary top strands at the stage is to reduce the required concrete strength at lifting.

For option 2, both the pretensioned and post-tensioned cases should be satisfied. This should not be a problem because the stability calculations are not sensitive to whether the temporary top strands are pretensioned or post-tensioned. In this example, for pretensioned temporary top strands, *FS* is 1.50 and *FS'* is 1.20 (step 5). For post-tensioned temporary top strands, *FS* is 1.51 and *FS'* is 1.21. The small differences derive from the increased effectiveness of the post-tensioned temporary top strands, which slightly reduce the girder camber and increase the compression in the top flange.

**Figure 10** and **Table 9** summarize the release and lifting requirements for the girders in this example. Similar information can be included in contract plans to provide the manufacturer with maximum flexibility in the production process.

### Step 7: Check girder shipping

This step will verify the number of temporary top strands required, if any, and will determine a range of allowable support locations and the concrete strength required for shipping. For this example, the six temporary top strands are assumed to be pretensioned with a concrete strength at release of 7.0 ksi (48 MPa).

**Table 9.** Summary of release and lifting requirements

| Girder | $N_t$ | Release strength $f'_{ci}$,* ksi | Lifting without temporary top strands† | | Lifting with temporary top strands ‡ | | Lifting with temporary top strands ** | |
|---|---|---|---|---|---|---|---|---|
| | | | $a_l$, ft | $f'_{ci}$, ksi | $a_l$, ft | $f'_{ci}$, ksi | $a_l$, ft | $f'_{ci}$, ksi |
| Spans 2 and 3 | 6 | 6.7 | 12.00 | 7.4 | 12.00 | 7.1 | 9.50 | 7.0 |

* This gives the minimum release strength for the girder sitting in the form only. Lifting strength must be achieved prior to lifting.

† Any required temporary top strands shall be stressed in the yard on the same day as release unless otherwise directed.

‡ Stripping is optional with temporary top strands post-tensioned prior to lifting in lieu of lifting without temporary top strands.

** Stripping is with pretensioned temporary top strands or post-tensioned temporary top strands stressed prior to lifting.

Note: $a_l$ = length of overhang for lifting; $f'_{ci}$ = required compressive strength of concrete at time of prestress transfer; $N_t$ = number of temporary top strands. 1 ft = 0.305 m; 1 ksi = 6.895 MPa.

The shipping analysis is heavily dependent on assumptions made about the trucking equipment, particularly the rotational stiffness of the tractor and trailer. Although shipping is ultimately the responsibility of the contractor, it is important for the designer to determine that the girders can be reasonably hauled. Whether this information is provided in the contract plans or is left for the contractor to determine is decided by the designer or in accordance with owner policy, practice, or procedure. However, if shipping information is provided in the contract plans, the specifications should require that the contractor verify the assumptions used in the analysis or provide alternative calculations for the shipment of the girders.

Truckers develop shipping schemes that best fit the capacity and geometry of their equipment. Overhangs at the tractor end can be limited by the distance between the front support and the cab. The tractor and trailer often have different hauling capacities, so overhangs are varied to distribute the load accordingly. Providing a range of shipping configurations allows the carrier the flexibility to plan efficient loads.

**Figure 11** and **Table 10** show the allowable trucking configurations determined for the girders in this example. Information is provided for both equal and unequal overhangs. Note that for unequal overhangs, the sum of the cantilevers must equal the single value given in the table. Other combinations of overhang lengths are possible but were not checked as part of the analysis. The calculations in the following are at the harp-point section with $l_t$ equal to 119.5 ft (36.4 m) and equal overhangs $a_t$ of 28 ft (8.5 m).

**Find time-dependent prestress losses at 10 days** The WSDOT standard specifications allow I-girders to be shipped as early as 10 days after the release of prestress. This is the critical age for shipment because the concrete strength will be lower and the prestressing force will be greater than at any other potential shipping age. In storage, the girders are supported near the ends during the development of time-dependent losses. Shrinkage, creep, and relaxation losses are estimated for the 10-day period as follows.[7]

The concrete shrinkage strain can be determined from the following equation.

$$\varepsilon_{sh} = k_s k_{hs} k_f k_{td}(0.48 \times 10^{-3})$$

where

$k_s$ = adjustment coefficient for volume-to-surface ratio

$k_{hs}$ = adjustment coefficient for humidity on shrinkage

$k_f$ = adjustment coefficient for concrete strength

$k_{td}$ = adjustment coefficient for development over time

$$k_s = 1.45 - 0.13(V/S) = 1.45 - 0.13(3.16) = 1.04$$

where

$V/S$ = volume-to-surface ratio of girder, as given in Table 2

$$k_{hs} = 2.0 - 0.014H = 2.0 - 0.014(80) = 0.88$$

where

$H$ = average annual ambient relative humidity

$$k_f = \frac{5}{1+f'_{ci}} = \frac{5}{1+7.0} = 0.625$$

$$k_{td} = \frac{t}{61-4f'_{ci}+t} = \frac{10}{61-4(7.0)+10} = 0.23$$

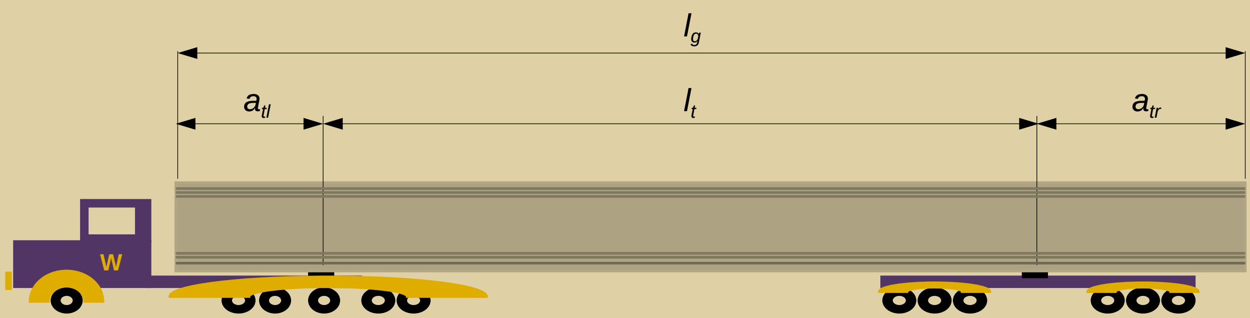



**Figure 11.** This diagram shows the trucking configuration used in conjunction with the shipping summary in Table 10. Note: $a_{tl}$ = left overhang length for shipping; $a_{tr}$ = right overhang length for shipping; $l_g$ = overall length of girder; $l_t$ = girder length between lifting embedments.

**Table 10.** Summary of shipping requirements

| Girder | $f'_c$, ksi | Equal cantilevers | | Unequal cantilevers | | |
|---|---|---|---|---|---|---|
| | | $a_{tmin}$, ft | $a_{tmax}$, ft | $a_{tmin}$,* ft | $a_{tmax}$,† ft | Sum‡ |
| Spans 2 and 3 | 8.9 | 21.00 | 28.00 | 10.00 | 32.00 | 42.00 |

* This indicates the minimum unequal cantilever to be used during shipping.

† This indicates the maximum unequal cantilever to be used during shipping.

‡ The sum of the left and right unequal cantilevers shall be equal to this value.

Note: $a_{tmax}$ = maximum length of overhang for shipping; $a_{tmin}$ = minimum length of overhang for shipping; $f'_c$ = specified compressive strength of concrete. 1 ft = 0.305 m; 1 ksi = 6.895 MPa.

where

$t$ = maturity of concrete (in days), defined as the age of concrete between time of loading for creep calculations or end of curing for shrinkage calculations and time being considered for analysis of creep or shrinkage effects

$\varepsilon_{sh}$ = (1.04)(0.88)(0.625)(0.23)(0.48 × $10^{-3}$) = 0.064 × $10^{-3}$

The transformed-section coefficient for time-dependent interaction between concrete and bonded prestressing steel over time $K_{id}$ can be calculated with the following equation.

$$K_{id} = \frac{1}{1+\frac{E_p}{E_{ci}}\left(\frac{A_{ps}}{A_g}\right)\left(1+\frac{A_g e_p^2}{I_g}\right)\left[1+0.7\psi_b\left(t_f,t_i\right)\right]}$$

where

$\psi_b(t_f, t_i)$ = girder creep coefficient at final due to loading introduced at transfer = 1.9 $k_s k_{hc} k_f k_{td} t_i^{-0.118}$

where

$k_{hc}$ = adjustment coefficient for humidity on creep = 1.56 – 0.008$H$ = 1.56 – 0.008(80) = 0.92

$k_{td}$ = 1.0 at final for purposes of calculating $K_{id}$

$\psi_b(t_f, t_i)$ = 1.9(1.04)(0.92)(0.625)(1.0)(1.0)$^{-0.118}$ = 1.14

$$K_{id} = \frac{1}{1+\frac{28{,}600}{5328}\left(\frac{14.11}{972}\right)\left[1+\frac{(972)(28.80)^2}{956{,}329}\right]\left[1+0.7(1.14)\right]}$$

= 0.80

The prestress loss due to shrinkage of girder concrete $\Delta f_{pSR}$ can now be determined.

$\Delta f_{pSR}$ = $\varepsilon_{sh} E_p K_{id}$ = (0.064 × $10^{-3}$)(28,600)(0.80) = 1.45 ksi (10.0 MPa)

The concrete stress at the centroid of prestressing force immediately after transfer $f_{cgp}$ can be determined from the following equation.

$$f_{cgp} = \frac{P_{pt}}{A_g} + \frac{P_{pt}e_p^2}{I_g} - \frac{M_g e_p}{I_g}$$

where

$M_g$ = 49,397 kip-in. (5581 kN-m) from step 2

$P_{pt}$ = $A_{ps}f_{pt}$

$A_{ps}$ = 14.11 from step 5

$f_{pt}$ = $f_{pbt}$ - $\Delta f_{pES}$

where

$f_{pbt}$ = 200.74 ksi (1384 MPa) from step 2

$\Delta f_{pES}$ = 18.22 ksi (126 MPa) from step 5 but not calculated in this example

$f_{pt}$ = 200.74 – 18.22 = 182.52 ksi (1258 MPa)

$P_{pt}$ = (14.11)(182.5) = 2574 kip (11,450 kN)

$$f_{cgp} = \frac{2574}{972} + \frac{2574(28.80)^2}{956,329} - \frac{49,397(28.80)}{956,329}$$

= 3.394 ksi (23.4 MPa)

The prestress loss due to creep of girder concrete $\Delta f_{pCR}$ can now be calculated.

$$\Delta f_{pCR} = \frac{E_p}{E_{ci}} f_{cgp}\psi_b(t_d, t_i)K_{id}$$

where

$\psi_b(t_d, t_i)$ = girder creep coefficient at shipping due to loading introduced at transfer = 1.9 $k_s k_{hc} k_f k_{td} t_i^{-0.118}$ = 1.9(1.04)(0.92)(0.625)(0.23)$(1.0)^{-0.118}$ = 0.26

$$\Delta f_{pCR} = \frac{28,600}{5328}(3.394)(0.26)(0.80)$$

= 3.82 ksi (26.3 MPa)

The prestress loss due to steel relaxation between transfer and shipping $\Delta f_{pR1}$ can be determined by the next equation.

$$\Delta f_{pR1} = \left[\frac{f_{pt}}{45}\left(\frac{\log(24t)}{\log(24t_i)}\right)\left(\frac{f_{pt}}{f_{py}} - 0.55\right)\right]\left[1 - \frac{3(\Delta f_{pSR} + \Delta f_{pCR})}{f_{pt}}\right]K_{id}$$

$$= \left\{\left(\frac{182.52}{45}\right)\left[\frac{\log(240)}{\log(24)}\right]\left(\frac{182.52}{243} - 0.55\right)\right\}\left[1 - \frac{3(1.45 + 3.82)}{182.52}\right](0.80)$$

= 1.03 ksi (7.10 MPa)

Total long-term prestress loss at 10 days $f_{ptl10}$ = $\Delta f_{pSR}$ + $\Delta f_{pCR}$ + $\Delta f_{pR1}$ = 1.45 + 3.82 + 1.03 = 6.30 ksi (43.4 MPa)

**Compute stresses in vertical girder including 20% impact** The prestressing force at the section under consideration at the time of shipping $P_s$ is calculated from the following equation.

$P_s$ = $A_{ps}f_{pe}$

where

$f_{pe}$ = effective tensile stress in prestressing strands at section under consideration after applicable time-dependent losses
= $f_{pt} - f_{ptl10}$

$f_{pe}$ = (182.52 – 6.30) = 176.22 ksi (1215 MPa)

$P_s$ = (14.11)(176.22) = 2486 kip (11,058 kN)

$$M_h = \frac{w}{2}\left(l_l x_l - x_l^2 - a_l^2\right)$$

$$= \frac{1.114}{2}\left[(119.5)(42.2) - (42.2)^2 - (28)^2\right](12)$$

= 16,560 kip-in. (1871 kN-m)

Stresses at harp-point section with +20% impact

$$f_t = \frac{P_s}{A_g} - \frac{P_s e_{pi}}{S_t} + \frac{1.2M_h}{S_t}$$

$$f_t = \frac{2486}{972} - \frac{(2486)(28.80)}{22,230} + \frac{(16,560)(1.2)}{22,230}$$

= 0.231 ksi (1.59 MPa)

**Table 11.** Shipping stresses in plumb girder with six pretensioned temporary top strands and impact

| Point | $x$, ft | $e_{pi}$, in. | $P_s$, kip | $M_g$, kip-in. | Impact +20% | | Impact -20% | |
|---|---|---|---|---|---|---|---|---|
| | | | | | $f_t$, ksi | $f_b$, ksi | $f_t$, ksi | $f_b$, ksi |
| Transfer | 3.00 | 11.96 | 2512 | -60 | 1.230 | 3.833 | 1.231 | 3.832 |
| Support | 28.00 | 18.22 | 2511 | -5239 | 0.242 | 4.742 | 0.337 | 4.655 |
| Harp | 70.20 | 28.80 | 2486 | 16,560 | 0.231 | 4.702 | -0.067 | 4.976 |
| Midspan | 87.75 | 28.80 | 2491 | 18,618 | 0.341 | 4.610 | 0.006 | 4.919 |
| Harp | 105.30 | 28.80 | 2486 | 16,560 | 0.231 | 4.702 | -0.067 | 4.976 |
| Support | 147.50 | 18.22 | 2511 | -5239 | 0.242 | 4.742 | 0.337 | 4.655 |
| Transfer | 172.50 | 11.96 | 2512 | -60 | 1.230 | 3.833 | 1.231 | 3.832 |

Note: $e_{pi}$ = eccentricity of combined prestressing force in permanent pretensioned strands and pretensioned or post-tensioned temporary top strands at section under consideration; $f_b$ = concrete stress in bottom fiber of girder section; $f_t$ = concrete stress in top fiber of girder section; $M_g$ = self-weight bending moment of girder at section under consideration; $P_s$ = prestressing force at section under consideration at time of shipping; $x$ = distance from girder end to section under consideration. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN; 1 ksi = 6.895 MPa.

$$f_b = \frac{P_s}{A_g} + \frac{P_s e_{pi}}{S_b} - \frac{1.2M_h}{S_b}$$

$$f_b = \frac{2486}{972} + \frac{(2486)(28.80)}{24{,}113} - \frac{(16{,}560)(1.2)}{24{,}113}$$

$$= 4.702 \text{ ksi (32.4 MPa)}$$

Stresses at harp-point section with -20% impact

$$f_t = \frac{P_s}{A_g} - \frac{P_s e_{pi}}{S_t} + \frac{0.8M_h}{S_t}$$

$$f_t = \frac{2486}{972} - \frac{(2486)(28.80)}{22{,}230} + \frac{(16{,}560)(0.8)}{22{,}230}$$

$$= -0.067 \text{ ksi (-0.46 MPa)}$$

$$f_b = \frac{P_s}{A_g} + \frac{P_s e_{pi}}{S_b} - \frac{0.8M_h}{S_b}$$

$$f_b = \frac{2486}{972} + \frac{(2486)(28.80)}{24{,}113} - \frac{(16{,}560)(0.8)}{24{,}113}$$

$$= 4.976 \text{ ksi (34.3 MPa)}$$

**Table 11** gives a complete list of stresses. The small tensile stress is within allowable limits, and the compressive stress at the harp point with -20% impact governs.

$$f_c' = \frac{f_b}{0.6} = \frac{4.976}{0.6} = 8.29 \text{ ksi (57.2 MPa)}$$

$$< 8.5 \text{ ksi (58.6 MPa)} \rightarrow \text{OK}$$

**Compute stability during shipping** For equipment currently available in Washington state, the allowable hauled weight per axle $W_{axle}$ is about 18 kip (80 kN), while the average rotational stiffness per axle $K_{ave}$ has been found to be about 4000 kip-in./rad (452 kN-m/rad). Table 1 lists additional shipping assumptions. For a beam with a total weight $W$ of 195.5 kip (870 kN), the number of axles required for truck shipment $N_a$ can be determined.

$$N_a = \frac{W}{W_{axle}} = \frac{195.5}{18} = 10.86$$

Therefore, it requires a minimum of 11 axles.

The sum of rotational spring constants of truck axles $K_\theta$ can be calculated.

$$K_\theta = N_a K_{ave} = (11)(4000)$$
$$= 44{,}000 \text{ kip-in./rad (4972 kN-m/rad)}$$

The radius of stability $r$ can be determined.

$$r = \frac{K_\theta}{W} = \frac{44{,}000}{195.5} = 225.1 \text{ in. (5718 mm)}$$

A typical truck supports the bottom of the girder at about 6 ft (1.8 m) above the roadway surface. Add to this the height from the bottom of the girder to its center of gravity.

$h_{cg}$ = (6)(12) + $y_b$ = (6)(12) + 39.66 = 111.66 in. (2836 mm)

where

$h_{cg}$ = height of center of gravity of girder above road

$y_b$ = height from bottom of girder to centroid of concrete section

$y$ = $h_{cg} - h_r$ = 111.66 – 24 = 87.66 in. (2227 mm)

where

$y$ = height of center of gravity of girder above roll axis

$h_r$ = height of roll center above road, taken as 24 in. (610 mm)

Increase $y$ by 2% to allow for camber. Then, $y$ is 89.41 in. (2271 mm).

$$F_{offset} = \left(\frac{l_t}{l_g}\right)^2 - \left(\frac{1}{3}\right) = \left(\frac{119.5}{175.5}\right)^2 - \left(\frac{1}{3}\right) = 0.13$$

where

$l_t$ = girder length between truck supports

$$e_{sweep} = \frac{1}{8}\left(\frac{175.5}{10}\right) = 2.19 \text{ in. (55.6 mm)}$$

where

$e_{sweep}$ = girder sweep tolerance during shipping, taken as $^1/_8$ in. per 10 ft (3 mm per 3 m) of girder length $l_g$

$e_{truck}$ = lateral placement tolerance on truck support, taken as 1 in. (25 mm)

$e_i$ = $e_{sweep}F_{offset} + e_{truck}$ = (2.19)(0.13) + 1 = 1.29 in. (32.6 mm)

$$E_c = 33{,}000K_1w_c^{1.5}\sqrt{f_c'} = 33{,}000(1.0)(0.155)^{1.5}\sqrt{8.5}$$

= 5871 ksi (40,481 MPa)

$$\bar{z}_o' = \frac{w}{12E_cI_yl_g}\left(\frac{1}{10}l_t^5 - a_t^2l_t^3 + 3a_t^4l_t + \frac{6}{5}a_t^5\right)$$

$$= \frac{1.114(12)^3}{12(5871)(71{,}914)(175.5)}\left[\frac{1}{10}(119.5)^5 - (28)^2(119.5)^3 + 3(28)^4(119.5) + \frac{6}{5}(28)^5\right]$$

= 2.90 in. (73.7 mm)

$$\theta = \frac{\alpha r + e_i}{r - y - \bar{z}_o} = \frac{(0.06)(225.1) + 1.29}{225.1 - 89.41 - 2.90} = 0.1114 \text{ rad}$$

where

$\theta$ = roll angle of major axis of girder with respect to plumb

$\alpha$ = superelevation angle or tilt angle of support

### Compute stresses of tilted girder at harp-point section, no impact

$$f_t = \frac{P_s}{A_g} - \frac{P_se_{pi}}{S_t} + \frac{M_h}{S_t}$$

$$f_t = \frac{2486}{972} - \frac{(2486)(28.80)}{22{,}230} + \frac{16{,}560}{22{,}230}$$

= 0.082 ksi (0.56 MPa)

$$f_b = \frac{P_s}{A_g} + \frac{P_se_{pi}}{S_b} - \frac{M_h}{S_b}$$

$$f_b = \frac{2486}{972} + \frac{(2486)(28.80)}{24{,}113} - \frac{16{,}560}{24{,}113}$$

= 4.840 ksi (33.4 MPa)

$M_l$ = $\theta M_h$ = 0.1114(16,560) = 1844 kip-in. (208 kN-m)

where

$M_l$ = lateral bending moment in tilted girder at section under consideration

**Table 12.** Shipping stresses with six pretensioned temporary top strands in tilted girder

| Point | *x*, ft | $M_g$, kip-in. | $M_l$, kip-in. | $f_{tu}$, ksi | $f_{bu}$, ksi | $f_{td}$, ksi | $f_{bd}$, ksi |
|---|---|---|---|---|---|---|---|
| Transfer | 3.00 | -60 | -7 | 1.228 | 3.830 | 1.223 | 3.834 |
| Support | 28.00 | -5239 | -583 | 0.091 | 4.543 | 0.488 | 4.854 |
| Harp | 70.20 | 16,560 | 1844 | 0.711 | 5.331 | -0.546 | 4.347 |
| Midspan | 87.75 | 18,618 | 2073 | 0.880 | 5.318 | -0.533 | 4.211 |
| Harp | 105.30 | 16,560 | 1844 | 0.711 | 5.331 | -0.546 | 4.347 |
| Support | 147.50 | -5239 | -583 | 0.091 | 4.543 | 0.488 | 4.854 |
| Transfer | 172.50 | -60 | -7 | 1.228 | 3.830 | 1.233 | 3.834 |

Note: $f_{bd}$ = concrete stress in bottom downhill fiber of tilted-girder section; $f_{bu}$ = concrete stress in bottom uphill fiber of tilted-girder section; $f_{td}$ = concrete stress in top downhill fiber of tilted-girder section; $f_{tu}$ = concrete stress in top uphill fiber of tilted-girder section; $M_g$ = self-weight bending moment of girder at section under consideration; $M_l$ = lateral bending moment in tilted girder at section under consideration; $x$ = distance from girder end to section under consideration. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN; 1 ksi = 6.895 MPa.

Top-fiber stress for uphill flange $f_{tu}$

$$= f_t + \frac{M_l\left(\frac{b_t}{2}\right)}{I_y} = 0.082 + \frac{1844\left(\frac{49.02}{2}\right)}{71{,}914}$$

= 0.711 ksi (4.90 MPa)

Top-fiber stress for downhill flange $f_{td}$

$$= f_t - \frac{M_l\left(\frac{b_t}{2}\right)}{I_y} = 0.082 - \frac{1844\left(\frac{49.02}{2}\right)}{71{,}914}$$

= -0.546 ksi (-3.77 MPa)

Bottom-fiber stress for uphill flange $f_{bu}$

$$= f_b + \frac{M_l\left(\frac{b_b}{2}\right)}{I_y} = 4.840 + \frac{1844\left(\frac{38.39}{2}\right)}{71{,}914}$$

= 5.331 ksi (36.8 MPa)

where

$b_b$ = bottom-flange width

Bottom-fiber stress for downhill flange $f_{bd}$

$$= f_b - \frac{M_l\left(\frac{b_b}{2}\right)}{I_y} = 4.840 - \frac{1844\left(\frac{38.39}{2}\right)}{71{,}914}$$

= 4.347 ksi (30.0 MPa)

For the maximum compressive stress in the bottom fiber of the uphill flange,

$$f'_c = \frac{f_{bu}}{0.6} = \frac{5.331}{0.6} = 8.89 \text{ ksi (61.3 MPa)} > 8.5 \text{ ksi}$$

(58.6 MPa) → no good

Therefore, the concrete strength required for shipping exceeds the design strength specified for final service conditions. Increasing the design strength to 8.9 ksi (61.4 MPa) and recalculating step 7 results in the list of stresses in the tilted girder (**Table 12**). Compression in the uphill bottom flange governs, while tension in the downhill top flange governs.

$$f'_c = \frac{f_{bu}}{0.6} = \frac{5.331}{0.6} = 8.89 \text{ ksi (61.3 MPa)}$$

< 8.9 ksi (61.4 MPa) → OK

$$f'_c = \left(\frac{f_{td}}{0.237}\right)^2 = \left(\frac{-0.546}{0.237}\right)^2 = 5.30 \text{ ksi (36.5 MPa)}$$

< 8.9 ksi (61.4 MPa) → OK

**Table 13.** Shipping stresses in plumb girder with six post-tensioned temporary top strands and impact

| Point | $x$, ft | $e_{pi}$, in. | $P_s$, kip | $M_g$, kip-in. | Impact +20% | | Impact -20% | |
|---|---|---|---|---|---|---|---|---|
| | | | | | $f_t$, ksi | $f_b$, ksi | $f_t$, ksi | $f_b$, ksi |
| Transfer | 3.00 | 11.82 | 2515 | -60 | 1.247 | 3.823 | 1.248 | 3.822 |
| Support | 28.00 | 18.04 | 2518 | -5239 | 0.264 | 4.735 | 0.358 | 4.648 |
| Harp | 70.20 | 28.46 | 2495 | 16,560 | 0.267 | 4.689 | -0.031 | 4.963 |
| Midspan | 87.75 | 28.45 | 2502 | 18,618 | 0.377 | 4.598 | 0.042 | 4.907 |
| Harp | 105.30 | 28.41 | 2497 | 16,560 | 0.272 | 4.688 | -0.026 | 4.962 |
| Support | 147.50 | 17.91 | 2524 | -5239 | 0.281 | 4.731 | 0.375 | 4.644 |
| Transfer | 172.50 | 11.69 | 2521 | -60 | 1.264 | 3.819 | 1.265 | 3.818 |

Note: $e_{pi}$ = eccentricity of combined prestressing force in permanent pretensioned strands and pretensioned or post-tensioned temporary top strands at section under consideration; $f_b$ = concrete stress in bottom fiber of girder section; $f_t$ = concrete stress in top fiber of girder section; $M_g$ = self-weight bending moment of girder at section under consideration; $P_s$ = prestressing force at section under consideration at time of shipping; $x$ = distance from girder end to section under consideration. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN; 1 ksi = 6.895 MPa.

**Table 14.** Shipping stresses with six post-tensioned temporary top strands in tilted girder

| Point | $x$, ft | $M_g$, kip-in. | $M_l$, kip-in. | $f_{tu}$, ksi | $f_{bu}$, ksi | $f_{td}$, ksi | $f_{bd}$, ksi |
|---|---|---|---|---|---|---|---|
| Transfer | 3.00 | -60 | -7 | 1.245 | 3.821 | 1.249 | 3.824 |
| Support | 28.00 | -5239 | -583 | 0.112 | 4.536 | 0.510 | 4.847 |
| Harp | 70.20 | 16,560 | 1844 | 0.746 | 5.318 | -0.511 | 4.334 |
| Midspan | 87.75 | 18,618 | 2073 | 0.916 | 5.306 | -0.496 | 4.200 |
| Harp | 105.30 | 16,560 | 1844 | 0.751 | 5.317 | -0.506 | 4.333 |
| Support | 147.50 | -5239 | -583 | 0.129 | 4.532 | 0.527 | 4.843 |
| Transfer | 172.50 | -60 | -7 | 1.262 | 3.816 | 1.267 | 3.820 |

Note: $f_{bd}$ = concrete stress in bottom downhill fiber of tilted-girder section; $f_{bu}$ = concrete stress in bottom uphill fiber of tilted-girder section; $f_{td}$ = concrete stress in top downhill fiber of tilted-girder section; $f_{tu}$ = concrete stress in top uphill fiber of tilted-girder section; $M_g$ = self-weight bending moment of girder at section under consideration; $M_l$ = lateral bending moment in tilted girder at section under consideration; $x$ = distance from girder end to section under consideration. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN; 1 ksi = 6.895 MPa.

**Tables 13** and **14** recreate Tables 11 and 12, respectively, except that post-tensioned temporary top strands are used in lieu of pretensioned temporary top strands. Again, it appears that post-tensioned temporary top strands are slightly more effective than their pretensioned counterparts.

## Compute factor of safety against cracking FS

$$f_r = 0.237\sqrt{f_c'} = 0.237\sqrt{8.9} = 0.708 \text{ ksi (4.88 MPa)}$$

$$f_t = 0.082 \text{ ksi (0.56 MPa)}$$

$$M_{lat} = \frac{2\left(f_r + f_t\right)I_y}{b_t} = \frac{2(0.708+0.082)(71{,}914)}{49.02}$$

$$= 2318 \text{ kip-in. (262 kN-m)}$$

$$\theta_{max} = \frac{M_{lat}}{M_h} = \frac{2318}{16{,}560} = 0.1400 \text{ rad}$$

$$FS = \frac{r\left(\theta_{max} - \alpha\right)}{\bar{z}_o\theta_{max} + e_i + y\theta_{max}}$$

$$= \frac{225.1(0.1400-0.06)}{2.83(0.1400)+1.29+89.41(0.1400)}$$

$$= 1.27 > 1.0 \rightarrow \text{OK}$$

**Table 15.** Shipping stresses in plumb girder with six pretensioned temporary top strands and impact with unequal overhangs

| Point | $x$, ft | $e_{pi}$, in. | $P_s$, kip | $M_g$, kip-in. | Impact +20% | | Impact -20% | |
|---|---|---|---|---|---|---|---|---|
| | | | | | $f_t$, ksi | $f_b$, ksi | $f_t$, ksi | $f_b$, ksi |
| Transfer | 3.00 | 11.96 | 2512 | -60 | 1.230 | 3.833 | 1.231 | 3.832 |
| Support | 10.00 | 13.71 | 2512 | -668 | 0.999 | 4.045 | 1.011 | 4.034 |
| Harp | 70.20 | 28.80 | 2486 | 26,035 | 0.743 | 4.230 | 0.274 | 4.662 |
| Midspan | 87.75 | 28.80 | 2491 | 24,701 | 0.669 | 4.307 | 0.225 | 4.717 |
| Harp | 105.30 | 28.80 | 2486 | 19,251 | 0.377 | 4.568 | 0.030 | 4.887 |
| Support | 143.50 | 19.22 | 2511 | -6843 | 0.043 | 4.925 | 0.166 | 4.812 |
| Transfer | 172.50 | 11.96 | 2512 | -60 | 1.230 | 3.833 | 1.231 | 3.832 |

Note: $e_{pi}$ = eccentricity of combined prestressing force in permanent pretensioned strands and pretensioned or post-tensioned temporary top strands at section under consideration; $f_b$ = concrete stress in bottom fiber of girder section; $f_t$ = concrete stress in top fiber of girder section; $M_g$ = self-weight bending moment of girder at section under consideration; $P_s$ = prestressing force at section under consideration at time of shipping; $x$ = distance from girder end to section under consideration. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN; 1 ksi = 6.895 MPa.

**Table 16.** Shipping stresses with six pretensioned temporary top strands in tilted girder with unequal overhangs

| Point | $x$, ft | $M_g$, kip-in. | $M_l$, kip-in. | $f_{tu}$, ksi | $f_{bu}$, ksi | $f_{td}$, ksi | $f_{bd}$, ksi |
|---|---|---|---|---|---|---|---|
| Transfer | 3.00 | -60 | -7 | 1.228 | 3.830 | 1.233 | 3.834 |
| Support | 10.00 | -668 | -78 | 0.978 | 4.019 | 1.032 | 4.061 |
| Harp | 70.20 | 26,035 | 3042 | 1.545 | 5.258 | -0.528 | 3.634 |
| Midspan | 87.75 | 24,701 | 2886 | 1.431 | 5.282 | -0.536 | 3.742 |
| Harp | 105.30 | 19,251 | 2249 | 0.970 | 5.328 | -0.563 | 4.127 |
| Support | 143.50 | -6483 | -799 | -0.168 | 4.655 | 0.377 | 5.082 |
| Transfer | 172.50 | -60 | -7 | 1.228 | 3.830 | 1.233 | 3.834 |

Note: $f_{bd}$ = concrete stress in bottom downhill fiber of tilted-girder section; $f_{bu}$ = concrete stress in bottom uphill fiber of tilted-girder section; $f_{td}$ = concrete stress in top downhill fiber of tilted-girder section; $f_{tu}$ = concrete stress in top uphill fiber of tilted-girder section; $M_g$ = self-weight bending moment of girder at section under consideration; $M_l$ = lateral bending moment in tilted girder at section under consideration; $x$ = distance from girder end to section under consideration. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN; 1 ksi = 6.895 MPa.

### Compute factor of safety against failure *FS'*

$$\theta'_{max} = \frac{z_{max} - h_r\alpha}{r} + \alpha = \frac{36 - 24(0.06)}{225.1} + 0.06$$

$$= 0.2135 \text{ rad}$$

where

$z_{max}$ = distance from centerline of vehicle to center of dual tires

$$\bar{z}'_o = \bar{z}_o\left(1 + 2.50\theta'_{max}\right) = 2.83\left[1 + 2.5(0.2135)\right]$$

$$= 4.35 \text{ in. (111 mm)}$$

$$FS' = \frac{r\left(\theta'_{max} - \alpha\right)}{\bar{z}'_o\theta'_{max} + e_i + y\theta'_{max}} = \frac{225.1(0.2135 - 0.06)}{\left[4.35(0.2135) + 1.29 + 89.41(0.2135)\right]}$$

$$= 1.62 > 1.50 \rightarrow \text{OK}$$

If no temporary top strands were used in the analysis and the lateral stability factors of safety are satisfied, the analysis is complete for the assumed shipping configuration. If pretensioned temporary top strands were used, the analysis should be repeated for post-tensioned temporary top strands. In this example, it was found that the post-tensioned temporary top strands slightly improved the factor of safety against cracking *FS* from 1.27 to 1.31. The factor of safety against failure *FS'* is unaffected by whether the strands are pretensioned or post-tensioned.

The entire analysis can be repeated for other shipping configurations, including cases where the overhangs used during shipping are not equal. This was done to find the range of limits in Table 10. Outside of the stated limits, either the lateral stability factors of safety were not adequate or the allowable concrete stresses were exceeded.

**Tables 15** and **16** list the stresses for the extreme configuration of unequal cantilevers, assuming that the temporary top strands were pretensioned. **Tables 17** and **18** show the same information, had the temporary top strands been post-tensioned. The sum total of 42 ft (12.8 m) shown in Table 10 was derived by doubling the minimum equal cantilever overhang. This minimum length of total overhang is required to provide adequate stability against overturning the truck. The overhangs were then varied until one of the limits described previously was exceeded.

For this example, the controlling case was in the tilted girder with pretensioned temporary top strands (Table 16). At the harp point near the maximum overhang ($x$ = 105.30 ft [32.1 m]), the compressive stress in the bottom flange on the uphill side of the tilt $f_{bu}$ is 5.328 ksi (36.74 MPa).

$$f_c' = \frac{f_{bu}}{0.6} = \frac{5.328}{0.6}$$

$$= 8.88 \text{ ksi (61.2 MPa)} < 8.9 \text{ ksi (61.4 MPa)} \rightarrow \text{OK}$$

**Table 17.** Shipping stresses in plumb girder with six post-tensioned temporary top strands and impact with unequal overhangs

| Point | $x$, ft | $e_{pi}$, in. | $P_s$, kip | $M_g$, kip-in. | Impact +20% | | Impact -20% | |
|---|---|---|---|---|---|---|---|---|
| | | | | | $f_t$, ksi | $f_b$, ksi | $f_t$, ksi | $f_b$, ksi |
| Transfer | 3.00 | 11.82 | 2515 | -60 | 1.247 | 3.823 | 1.248 | 3.822 |
| Support | 10.00 | 13.57 | 2516 | -668 | 1.017 | 4.037 | 1.029 | 4.026 |
| Harp | 70.20 | 28.46 | 2495 | 26,035 | 0.778 | 4.217 | 0.310 | 4.649 |
| Midspan | 87.75 | 28.45 | 2502 | 24,701 | 0.706 | 4.296 | 0.261 | 4.705 |
| Harp | 105.30 | 28.41 | 2497 | 19,251 | 0.417 | 4.554 | 0.070 | 4.873 |
| Support | 143.50 | 18.91 | 2523 | -6843 | 0.081 | 4.915 | 0.204 | 4.801 |
| Transfer | 172.50 | 11.69 | 2521 | -60 | 1.264 | 3.819 | 1.265 | 3.818 |

Note: $e_{pi}$ = eccentricity of combined prestressing force in permanent pretensioned strands and pretensioned or post-tensioned temporary top strands at section under consideration; $f_b$ = concrete stress in bottom fiber of girder section; $f_t$ = concrete stress in top fiber of girder section; $M_g$ = self-weight bending moment of girder at section under consideration; $P_s$ = prestressing force at section under consideration at time of shipping; $x$ = distance from girder end to section under consideration. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN; 1 ksi = 6.895 MPa.

**Table 18.** Shipping stresses with six post-tensioned temporary top strands in tilted girder with unequal overhangs

| Point | $x$, ft | $M_g$, kip-in. | $M_l$, kip-in. | $f_{tu}$, ksi | $f_{bu}$, ksi | $f_{td}$, ksi | $f_{bd}$, ksi |
|---|---|---|---|---|---|---|---|
| Transfer | 3.00 | -60 | -7 | 1.245 | 3.821 | 1.249 | 3.824 |
| Support | 10.00 | -668 | -78 | 0.996 | 4.011 | 1.050 | 4.052 |
| Harp | 70.20 | 26,035 | 3042 | 1.580 | 5.245 | -0.493 | 3.621 |
| Midspan | 87.75 | 24,701 | 2886 | 1.467 | 5.271 | -0.500 | 3.730 |
| Harp | 105.30 | 19,251 | 2149 | 1.010 | 5.314 | -0.523 | 4.113 |
| Support | 143.50 | -6843 | -799 | -0.130 | 4.645 | 0.415 | 5.072 |
| Transfer | 172.50 | -60 | -7 | 1.262 | 3.816 | 1.267 | 3.820 |

Note: $f_{bd}$ = concrete stress in bottom downhill fiber of tilted-girder section; $f_{bu}$ = concrete stress in bottom uphill fiber of tilted-girder section; $f_{td}$ = concrete stress in top downhill fiber of tilted-girder section; $f_{tu}$ = concrete stress in top uphill fiber of tilted-girder section; $M_g$ = self-weight bending moment of girder at section under consideration; $M_l$ = lateral bending moment in tilted girder at section under consideration; $x$ = distance from girder end to section under consideration. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN; 1 ksi = 6.895 MPa.

Because any longer overhang would cause this stress to exceed the allowable limit, the limits of 10 ft to 32 ft (3.05 m to 9.75 m) were chosen (Table 10).

## Step 8: Check girder stresses after erection

This step does not cover the actual girder erection but the stresses in the in-place girder immediately after the temporary top strands are cut. Assuming that the same lifting embedments are used for erection as were used for stripping, stresses during erection will be less critical than at stripping.

The WSDOT standard specifications require that the girders be braced against tipping before the temporary top strands are cut. The temporary top strands must then be cut before placing the intermediate diaphragms so no additional dead load is acting on the girders at this stage. This can potentially be problematic for relatively short, heavily stressed girders where the temporary top strands were used primarily to control stresses. This step ensures that the allowable temporary concrete stresses are not exceeded at this stage.

Assume that the girders are erected and braced and the temporary top strands are cut at 10 days from release of prestress, which is a worst-case scenario. Check the stresses at the harp-point section, assuming the temporary top strands were pretensioned. The long-term prestress loss is the same as at shipping. From step 7, the following values can be calculated.

$f_{pe} = f_{pt} - \Delta f_{pLT} = 182.52 - 6.30 = 176.22$ ksi (1215 MPa)

where

$f_{pe}$ = effective tensile stress in prestressing strands at section under consideration after applicable time-dependent losses

$\Delta f_{pLT}$ = sum of long-term prestress losses at time considered = $f_{ptl10}$ in this case

$P_s = A_{ps} f_{pe} = (14.11)(176.22) = 2486$ kip (11,058 kN)

For the six temporary top strands only (the force is negative because the strands are being cut),

$P_t = N_t A_p f_{pe} = -6(0.217)(176.22) = -229.4$ kip (-1021 kN)

where

$P_t$ = prestressing force in temporary top strands at section under consideration after erection

$e_{pt} = 2 - y_t = 2 - 43.02 = -41.02$ in. (-1042 mm)

The change in concrete stress at the level of the permanent prestressing can be calculated.

$$\Delta f_{cgpt} = \frac{P_t}{A_g} + \frac{P_t e_{pt} e_{pp}}{I_g} = \frac{-229.4}{972} + \frac{-229.4(-41.02)(35.90)}{956{,}329}$$

$$= 0.117 \text{ ksi } (0.81 \text{ MPa})$$

This represents an increase in compression in the concrete at the level of the permanent pretensioning, which results in a net loss of prestress. The effective prestress in the permanent prestressing steel is then calculated.

$$E_c = 33{,}000 K_1 w_c^{1.5} \sqrt{f'_c} = 33{,}000(1.0)(0.155)^{1.5}\sqrt{8.9}$$

$$= 6008 \text{ ksi } (41{,}424 \text{ MPa})$$

$$f'_{pe} = f_{pe} - \Delta f_{cgpt}\left(\frac{E_p}{E_c}\right) = 176.22 - 0.117\left(\frac{28{,}600}{6008}\right)$$

$$= 175.66 \text{ ksi } (1211 \text{ MPa})$$

where

$f'_{pe}$ = effective tensile stress in the permanent pretensioned strands after cutting the temporary top strands

The effective prestressing force in the permanent pretensioned strands after cutting the temporary top strands $P_e$ is then calculated.

$P_e = A_{ps} f'_{pe} = (12.80)(175.66) = 2249$ kip (10,004 kN)

The temporary oak block supports are located $10^1/_2$ in. (270 mm) from the girder ends.

$$M_h = \frac{w}{2}\left(l_{span} x_l - x_l^2 - a_l^2\right) = \frac{1.114}{2}\Big[173.75(69.325) - (69.325)^2 - (0.875)^2\Big](12)$$

$$= 48{,}371 \text{ kip-in. } (5465 \text{ kN-m})$$

$$f_t = \frac{P_e}{A_g} - \frac{P_e e_h}{S_t} + \frac{M_h}{S_t}$$

$$f_t = \frac{2249}{972} - \frac{(2249)(35.90)}{22{,}230} + \frac{48{,}371}{22{,}230}$$

$$= 0.858 \text{ ksi } (5.91 \text{ MPa})$$

**Table 19.** Stresses in erected girder after six pretensioned temporary top strands are cut

| Point | $x$, ft | $e_p$, in. | $P_e$, kip | $M_g$, kip-in. | $f_t$, ksi | $f_b$, ksi |
|---|---|---|---|---|---|---|
| Support | 0.875 | 16.76 | 666 | -5 | 0.183 | 1.149 |
| Transfer | 3.00 | 17.34 | 2284 | 2432 | 0.677 | 3.892 |
| Harp | 70.20 | 35.90 | 2249 | 48,371 | 0.858 | 3.656 |
| Midspan | 87.75 | 35.90 | 2253 | 50,429 | 0.948 | 3.582 |
| Harp | 105.30 | 35.90 | 2249 | 48,371 | 0.858 | 3.656 |
| Transfer | 172.50 | 17.34 | 2284 | 2432 | 0.677 | 3.892 |
| Support | 174.625 | 16.76 | 666 | -5 | 0.183 | 1.149 |

Note: $e_p$ = eccentricity of prestressing force at section under consideration; $f_b$ = concrete stress in bottom fiber of girder section; $f_t$ = concrete stress in top fiber of girder section; $M_g$ = self-weight bending moment of girder at section under consideration; $P_e$ = effective prestressing force in permanent pretensioned strands at section under consideration after erection and cutting of temporary top strands; $x$ = distance from girder end to section under consideration. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN; 1 ksi = 6.895 MPa.

**Table 20.** Stresses in erected girder after six post-tensioned temporary top strands are cut

| Point | $x$, ft | $e_p$, in. | $P_e$, kip | $M_g$, kip-in. | $f_t$, ksi | $f_b$, ksi |
|---|---|---|---|---|---|---|
| Support | 0.875 | 16.76 | 665 | -5 | 0.183 | 1.147 |
| Transfer | 3.00 | 17.34 | 2281 | 2432 | 0.676 | 3.886 |
| Harp | 70.20 | 35.90 | 2246 | 48,371 | 0.860 | 3.649 |
| Midspan | 87.75 | 35.90 | 2251 | 50,429 | 0.949 | 3.576 |
| Harp | 105.30 | 35.90 | 2246 | 48,371 | 0.860 | 3.649 |
| Transfer | 172.50 | 17.34 | 2281 | 2432 | 0.676 | 3.886 |
| Support | 174.625 | 16.76 | 665 | -5 | 0.183 | 1.147 |

Note: $e_p$ = eccentricity of prestressing force at section under consideration; $f_b$ = concrete stress in bottom fiber of girder section; $f_t$ = concrete stress in top fiber of girder section; $M_g$ = self-weight bending moment of girder at section under consideration; $P_e$ = effective prestressing force in permanent pretensioned strands at section under consideration after erection and cutting of temporary top strands; $x$ = distance from girder end to section under consideration. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN; 1 ksi = 6.895 MPa.

$$f_b = \frac{P_e}{A_g} + \frac{P_e e_h}{S_b} - \frac{M_h}{S_b}$$

$$f_b = \frac{2249}{972} + \frac{(2249)(35.90)}{24{,}113} - \frac{48{,}371}{24{,}113}$$

$$= 3.656 \text{ ksi (25.21 MPa)}$$

**Table 19** shows the complete list of stresses along the girder length, while **Table 20** lists the stresses if the temporary top strands had been post-tensioned. The stresses are essentially the same regardless of whether the temporary top strands were pretensioned or post-tensioned.

In this case, the controlling section is located at the end of the transfer length.

$$f_c' = \frac{f_b}{0.6} = \frac{3.892}{0.6}$$

$$= 6.49 \text{ ksi (44.7 MPa)} < 8.9 \text{ ksi (61.4 MPa)} \rightarrow \text{OK}$$

The temporary stress of 0.60 $f_c'$ is still allowed in this case. Once the superstructure is complete, a separate check of girder stresses, including the effective prestress and all permanent loads, live load, and one-half the sum of effective prestress and permanent loads, should be made and compared to allowable compressive stresses of 0.45 $f_c'$ and 0.40 $f_c'$, respectively.

## Recommendations

### Information included on the contract plans

As has been demonstrated, the lifting analysis is an important aspect of design because it establishes the optimum configuration of the permanent prestressing and the required concrete strength at release. The shipping analysis is also important because it generally establishes the required number of temporary top strands and can govern the final design concrete strength. These variables can affect the calculation of girder camber, which in turn affects the design for final service conditions.

There has been much debate within the industry on how much of this information should be shown in the contract plans. Many feel that lifting and shipping are solely the responsibility of the contractor and its subcontractors or material suppliers. The authors believe that it is not quite that simple. If the girders are not optimized for fabrication, handling, and shipping in the design phase and the contractor wants or needs to do so after the bid, the changes proposed may affect girder camber and other aspects of the design for final service conditions. It is always desirable to avoid changes after the bid.

For lifting, the method of analysis has been well established and the designer can specify the parameters within predetermined limits. For example, WSDOT and industry have established limits on concrete release strength, the maximum number of temporary top strands, the maximum number of total strands, and the maximum girder weight. These limits are occasionally updated as industry improves its capabilities in these areas. If a specific design falls within these limits, the designer can be assured that the design is viable. If a design falls outside of these limits, the designer can consult with industry on that specific project.

WSDOT's current practice is to show the lifting configuration only for the temporary top strands specified in the contract plans, assuming that these strands are pretensioned (step 5). Of course, if no temporary top strands are specified, the lifting configuration considers none. WSDOT is considering including the other release and lifting options shown in Table 9 when temporary top strands are required. The authors believe that disclosing this information on the contract plans will provide maximum flexibility to the manufacturer, which should lead to increased efficiency and reduced costs.

One possible exception to this is the concrete strength at release with the girder sitting in the form (step 3). There can be quite a disparity between the concrete strength at release and the required concrete strength at lifting. In this example, the minimum required release strength (step 3) and the strength required for lifting without temporary top strands (step 2) are 6.7 ksi and 7.4 ksi (46 MPa and 51 MPa), respectively, which is a difference of 0.7 ksi (4.8 MPa). It is not conservative to assume that this difference can be made up in a one- to two-hour period after the discontinuation of accelerated curing.

More typically, manufacturers consider releasing the prestress at a concrete strength less than that required for lifting only when the measured strength is close to the required strength. Therefore, it may be more appropriate to specify a two-tiered approach.

The first tier is to allow a lower-bound tolerance on the concrete release strength (for example, -3% or -200 psi [-1380 kPa]) with the knowledge that the concrete strength will continue to increase until the girder is lifted. This approach will not require additional cylinders to be tested. Even if the concrete gains no additional strength—a worst-case scenario—the overstress at lifting will be slight. After all, it has been consistently shown that concrete compressive stresses slightly greater than the specified 0.60 $f_c'$ cause no distress in early-age concrete.[10]

The second tier would be to allow the manufacturer to release the prestress at the concrete strength calculated in step 3. This approach would require additional cylinder testing prior to lifting. Manufacturers can assess their risk by collecting statistical data on the strength gain between the discontinuation of accelerated curing and lifting. Also, a plan for extending the curing cycle, if necessary, should be established and approved. It should be clear in the contract documents that achieving the required concrete strength at lifting is solely the responsibility of the manufacturer.

Shipping is strongly dependent on the contractor's equipment and the route to the jobsite. WSDOT performs shipping analyses based on the fundamental criteria listed in the WSDOT *Bridge Design Manual*[3], which are the same as previously listed in this paper. At this time, WSDOT shows only the minimum equal overhang option (Fig. 11 and Table 10) on the contract plans. In order to use these trucking support locations, the standard specifications require the contractor to verify that its equipment meets or exceeds these minimum criteria. Otherwise, the contractor is required to submit an alternative shipping plan.

The available delivery routes can also determine whether a single-piece pretensioned girder is a viable option for a specific jobsite location. When single-piece girders exceed a certain size limit, WSDOT provides alternative designs for pretensioned, single-piece, and spliced post-tensioned girder solutions. The alternatives allow the most competitive bidding for any jobsite location.

Ultimately, what is shown on the contract plans will depend on the comfort level of the contracting agency.

The most important aspect to be taken from this paper is the optimization of the permanent pretensioned strand configuration. The lifting and shipping options presented are generally less critical but gain importance as the girder sections are stretched to their practical limits.

## Conclusion

A step-by-step design procedure has been presented to optimize the design of pretensioned concrete girders for maximum production efficiency. The procedure combines the analysis for safe handling and shipping with the fabricator's production scheduling and efficiency needs. By properly proportioning the straight and harped strands to result in the lowest possible exit location of the prestressing force at the girder ends, demands on the stressing bed are minimized. The analytical procedure avoids adverse affects on any other aspects of the design and avoids the need for debonded strands, which unnecessarily weaken the girder ends in both flexure and shear. The procedure also results in the least required concrete strengths for the selected lifting and shipping scenarios.

While the iterative design procedures may seem onerous, properly designed software can provide a solution within a matter of minutes. WSDOT's prestressed concrete girder design software, PGSuper for pretensioned girders and PGSplice for post-tensioned, spliced girders, has been updated to handle the analytical procedures described in this paper. This software can be freely downloaded from WSDOT's website at www.wsdot.wa.gov/eesc/bridge/software. Spreadsheets have also been developed for this purpose.

## Disclaimer

The opinions and conclusions expressed in this paper are those of the authors and are not necessarily those of the Washington State Department of Transportation.

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## Notation

$a_l$ = length of overhang for lifting

$a_t$ = length of equal overhangs for shipping

$a_{tl}$ = left overhang length for shipping

$a_{tmax}$ = maximum length of overhang for shipping

$a_{tmin}$ = minimum length of overhang for shipping

$a_{tr}$ = right overhang length for shipping

$A_g$ = gross area of concrete section

$A_p$ = area of one prestressing strand

$A_{ps}$ = total area of prestressing steel in concrete section

$b$ = distance from end of girder to harp point

$b_b$ = bottom-flange width

$b_t$ = top-flange width

$D_{40}$ = estimated girder camber at 40 days after casting (lower bound)

$D_{120}$ = estimated girder camber at 120 days after casting (upper bound)

$e'$ = change in eccentricity of prestressing steel area between harp point and end of girder = $e_h - e_e$

$e_e$ = eccentricity of total area of prestressing steel at end of girder

$e_h$ = eccentricity of prestressing force at harp-point section

$e_i$ = initial eccentricity of center of gravity of girder from the roll axis

$e_{lift}$ = lateral placement tolerance for lifting devices

$e_p$ = eccentricity of prestressing force at section under consideration

$e_P$ = eccentricity of prestressing force from center of bed overturning

$e_{pi}$ = eccentricity of combined prestressing force in permanent pretensioned strands and pretensioned or post-tensioned temporary top strands at section under consideration

$e_{pp}$ = eccentricity of prestressing force in permanent pretensioned strands

$e_{pt}$ = eccentricity of prestressing force in pretensioned or post-tensioned temporary top strands

$e_{sweep}$ = girder sweep tolerance, taken as $^1/_{16}$ in. per 10 ft of girder length for lifting and $^1/_8$ in. per 10 ft of girder length for shipping

$e_t$ = eccentricity of prestressing force at transfer-length section

$e_{truck}$ = lateral placement tolerance on truck support

$e_W$ = eccentricity of abutment weight from center of bed overturning

$E_{ci}$ = modulus of elasticity of concrete at release strength

$E_p$ = modulus of elasticity of prestressing steel

$f_b$ = concrete stress in bottom fiber of girder section

$f_{bd}$ = concrete stress in bottom downhill fiber of tilted-girder section

$f_{bu}$ = concrete stress in bottom uphill fiber of tilted-girder section

$f_c$ = compressive stress in concrete

$f_c'$ = specified compressive strength of concrete

$f_{cgp}$ = concrete stress at centroid of prestressing force immediately after transfer

$f_{ci}'$ = required compressive strength of concrete at time of prestress transfer

$f_{pbt}$ = tensile stress in pretensioned strands immediately before transfer

$f_{pD}$ = initial tensile stress in post-tensioned temporary top strand at dead end after seating

$f_{pe}$ = effective tensile stress in prestressing strands at section under consideration after applicable time-dependent losses

$f_{pe}'$ = effective tensile stress in the permanent pretensioned strands after cutting the temporary top strands

$f_{pj}$ = initial tensile stress in prestressing strands after jacking

$f_{pL}$ = initial tensile stress in post-tensioned temporary top strand at jacking end after seating

$f_{pmax}$ = maximum tensile stress in post-tensioned temporary top strand after seating but before elastic losses due to subsequent jacking

$f_{pp}$ = tensile stress in pretensioned permanent strands at section under consideration after transfer plus jacking and seating of all temporary top strands

$f_{pt}$ = tensile stress in pretensioned strands at section under consideration immediately after transfer

$f_{ptave}$ = average tensile stress in post-tensioned temporary top strands at section under consideration after jacking and seating of all temporary top strands

$f_{ptl10}$ = total long-term prestress loss at 10 days

$f_{ptmax}$ = tensile stress in first post-tensioned temporary top strand at section under consideration after seating

$f_{ptmin}$ = tensile stress in first post-tensioned temporary top strand at section under consideration after jacking and seating of all temporary top strands

$f_{py}$ = specified yield strength of prestressing steel

$f_r$ = modulus of rupture of concrete

$f_t$ = concrete stress in top fiber of girder section

$f_{td}$ = concrete stress in top downhill fiber of tilted-girder section

$f_{tu}$ = concrete stress in top uphill fiber of tilted-girder section

$F_b$ = distance from bottom of girder to lowest harped-strand bundle at midspan

$F_{offset}$ = offset factor that determines the distance between the roll axis and the center of gravity of the arc of a curved girder

$FS$ = lateral stability factor of safety against cracking

$FS'$ = lateral stability factor of safety against failure

$h_{cg}$ = height of center of gravity of girder above road

$h_r$ = height of roll center above road

$H$ = average annual ambient relative humidity

$I_g$ = gross major-axis moment of inertia

$I_y$ = gross minor-axis (lateral) moment of inertia

$k_f$ = adjustment coefficient for concrete strength

$k_{hc}$ = adjustment coefficient for humidity on creep

$k_{hs}$ = adjustment coefficient for humidity on shrinkage

$k_s$ = adjustment coefficient for volume-to-surface ratio

$k_{td}$ = adjustment coefficient for development over time

$K$ = selected factor of safety for stressing-bed design

$K_1$ = factor to adjust for aggregate stiffness

$K_{ave}$ = average rotational stiffness per axle

$K_{id}$ = transformed-section coefficient for time-dependent interaction between concrete and bonded prestressing steel over time

$K_w$ = post-tensioning wobble-friction factor

$K_\theta$ = sum of rotational spring constants of truck axles

$l_g$ = length of girder

$l_l$ = girder length between lifting embedments

$l_s$ = length over which anchorage seating affects prestressing force

$l_{span}$ = span length, center to center of bearings

$l_t$ = girder length between truck supports

$M_g$ = self-weight bending moment of girder at section under consideration

$M_h$ = self-weight bending moment of girder at harp-point section

$M_l$ = lateral bending moment in tilted girder at section under consideration

$M_{lat}$ = lateral bending moment of girder at cracking

$M_r$ = resisting moment of stressing-bed abutment

$N$ = total number of permanent pretensioning strands

$N_a$ = number of axles required for truck shipment

$N_t$ = number of temporary top strands

$P_e$ = effective prestressing force in permanent pretensioned strands at section under consideration after erection and cutting of temporary top strands

$P_{jack}$ = jacking force resisted by stressing bed

$P_{pp}$ = prestressing force in pretensioned permanent strands at section under consideration after transfer plus jacking and seating of all temporary top strands

$P_{pt}$ = prestressing force at section under consideration immediately after transfer

$P_{ptave}$ = average total prestressing force in post-tensioned temporary top strands at section under consideration after jacking and seating

$P_{ptmax}$ = prestressing force in first post-tensioned temporary top strand at section under consideration after seating

$P_r$ = axial resistance of stressing bed

$P_s$ = prestressing force at section under consideration at time of shipping

$P_t$ = prestressing force in temporary top strands at section under consideration after erection

$r$ = radius of stability

$S_b$ = major-axis bottom-section modulus

$S_t$ = major-axis top-section modulus

$t$ = maturity of concrete (in days), defined as the age of concrete between time of loading for creep calculations, or end of curing for shrinkage calculations, and time being considered for analysis of creep or shrinkage effects

$t_i$ = assumed time between jacking and transfer of prestress

$V/S$ = volume-to-surface ratio of girder

$w$ = weight per unit length of girder

$w_c$ = density of concrete

$W$ = total weight of girder

$W_{abut}$ = weight of portion of stressing abutment resisting overturning

$W_{axle}$ = Washington state's allowable hauled weight per axle

$x$ = distance from girder end to section under consideration

$x_l$ = distance from lifting point or support to harp point

$y$ = height of center of gravity of girder above roll axis

$y_b$ = height from bottom of girder to centroid of concrete section

$y_r$ = height of roll axis above center of gravity of hanging girder

$y_t$ = height from top of girder to centroid of concrete section

$z_{max}$ = distance from centerline of vehicle to center of dual tires

$\bar{z}_0$ = theoretical lateral deflection of center of gravity of girder with full dead weight applied laterally

$\bar{z}_0'$ = theoretical lateral deflection of center of gravity of girder with full dead weight applied laterally, computed using effective moment of inertia for tilt angle $\theta$ under consideration

$\alpha$ = superelevation angle or tilt angle of support

$\gamma_c$ = density of concrete including reinforcement

$\Delta$ = total camber

$\Delta_{ohang}$ = additional component of upward deflection due to girder overhangs beyond lift points

$\Delta_{ps}$ = component of upward deflection due to prestress

$\Delta_{self}$ = component of downward deflection due to self-weight

$\Delta f_{cgpt}$ = elastic change of stress in permanent pretensioned strands due to cutting temporary top strands after erection

$\Delta f_{pCR}$ = prestress loss due to creep of girder concrete

$\Delta f_{pES}$ = prestress loss due to elastic shortening of girder concrete

$\Delta f_{pF}$ = prestress loss due to friction along length of post-tensioned strand

$\Delta f_{pLT}$ = sum of long-term prestress losses at time considered

$\Delta f_{pp}$ = elastic change of stress in permanent pretensioned strands due to post-tensioning of temporary top strands

$\Delta f_{pR0}$ = prestress loss due to steel relaxation between jacking and transfer

$\Delta f_{pR1}$ = prestress loss due to steel relaxation between transfer and shipping

$\Delta f_{pS}$ = prestress loss at jacking end due to seating of post-tensioned strand

$\Delta f_{pSR}$ = prestress loss due to shrinkage of girder concrete

$\Delta f_{pt}$ = elastic loss of stress in a temporary top strand due to post-tensioning of a subsequent temporary top strand

$\Delta l$ = assumed seating slip at post-tensioning anchorages

$\varepsilon_{sh}$ = concrete shrinkage strain

$\theta$ = roll angle of major axis of girder with respect to plumb

$\theta_i$ = initial roll angle of rigid girder measured from plumb

$\theta_{max}$ = tilt angle at which cracking begins measured from plumb

$\theta'_{max}$ = tilt angle at maximum factor of safety against failure measured from plumb

$\phi$ = strength-reduction or resistance factor

$\psi_b(t_d, t_i)$ = girder creep coefficient at shipping due to loading introduced at transfer

$\psi_b(t_f, t_i)$ = girder creep coefficient at final conditions due to loading introduced at transfer

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## Synopsis

The Washington State Department of Transportation (WSDOT) is updating its design methodology and detailing practices to facilitate optimized fabrication of precast, pretensioned concrete bridge girders. The objectives are reducing cost by saving on materials and labor, improving schedules by optimizing plant usage and production efficiencies, and enhancing quality by avoiding interferences and tight tolerances. Primary among these optimization techniques is the design and detailing of the permanent pretensioned strand configuration.

This paper presents a step-by-step design procedure and example problem that optimizes the pretensioning configuration for maximum production efficiency while maintaining compliance with the applicable requirements for safe handling and shipping. The procedure provides the manufacturer with a high degree of flexibility in plant usage and production turnover. Other optimization techniques, while important, are not covered in this paper.

## Keywords

Design, fabrication, girder, optimization, stressing bed.

## Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

## Reader comments

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